# West Sacramento Project

Yolo County, California
General Reevaluation Report

Appendix E - Geotechnical

West Sacramento Area Flood Control Agency

Central Valley Flood Protection Board

US Army Corps of Engineers
Sacramento District

# WEST SACRAMENTO PROJECT

# GENERAL REEVALUATION REPORT
# GEOTECHNICAL APPENDIX

**US Army Corps of Engineers**
**Sacramento District**

**PREPARED BY**

**GEOTECHNICAL ENGINEERING BRANCH**

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## ABBREVIATIONS

| | |
|---|---|
| ACN | Arcade Creek North |
| ACS | Arcade Creek South |
| ARCF | American River Common Features |
| ARFCD | American River Flood Control District |
| ARFCP | American River Flood Control Project |
| ARN | American River North Basin |
| ARS | American River South Basin |
| ASTM | American Society of Testing and Materials |
| ARWI | American River Watershed Investigation |
| BGS | Below Ground Surface |
| BTA | Blanket Theory Analysis |
| CB | Cement-Bentonite |

| | |
|---|---|
| CFS | Cubic Feet Per Second |
| CHP | California Highway Patrol |
| COS | City of Sacramento |
| CPT | Cone Penetrometer Test |
| CW | Cutoff Wall |
| CVFPB | Central Valley Flood Protection Board |
| CVFPP | Central Valley Flood Protection Plan |
| CY | Cubic Yard(s) |
| DBH | Diameter at Breast Height |
| DCN | Dry Creek North |
| DCS | Dry Creek South |
| DMM | Deep Mix Method |
| DSM | Deep Soil Mixing |
| DWR | Department of Water Resources |
| DWSC | Deep Water Ship Channel |
| DWSCWL | Deep Water Ship Channel West Levee |
| EFA | Erosion Function Apparatus |
| EIP | Early Implementation Project |
| EM | Engineering Manual |
| ETL | Engineering Technical Letter |
| EVS | Environmental Visualization System |
| FEMA | Federal Emergency Management Agency |
| FOS | Factor(s) Of Safety |
| FOSM | First Order Second Moment |
| FT | Foot/Feet |
| FT/S | Feet Per Second |
| GER | Geotechnical Engineering Report |
| GIS | Geographical Information System |
| GMS | Groundwater Modeling Software |
| GRR | General Reevaluation Report |
| H:V | Horizontal To Vertical Ratio |
| HTRW | Hazardous, Toxic, and Radioactive Waste |
| HQUSACE | Headquarters U.S. Army Corps Of Engineers |
| IBC | International Building Code |
| IWM | In-Stream Woody Material |
| JET | Jet Erosion Test |
| K | Coefficient Of Permeability |
| $K_H$ | Horizontal Hydraulic Conductivity Under Fully Saturated Conditions |
| $K_H/K_V$ | Ratio Between The Vertical And Horizontal Conductivities; Anisotropic Ratio |
| $K_V$ | Vertical Hydraulic Conductivity Under Fully Saturated Conditions |
| Ky | Yield Acceleration |
| LAR | Lower American River |
| LiDAR | Light Detection and Ranging |
| LM | Levee Mile |
| MA | Maintenance Area |
| MCDC | Magpie Creek Diversion Canal |

| | |
|---|---|
| MCY | Million Cubic Yards |
| MSWL | Mean Summer Water Level |
| MUSYM | Map Unit Symbol |
| Mw | Moment Magnitude |
| NAD83 | North American Datum of 1983 |
| NAFCI | Natomas Area Flood Control Improvement |
| NALP | North Area Levee Project0 |
| NAT | Natomas Basin |
| NAVD88 | North American Vertical Datum of 1988 |
| NCC | Natomas Cross Canal |
| NCEER | National Center for Earthquake Engineering Research |
| NCMWC | Natomas Central Mutual Water Company |
| NCSS | National Cooperative Soil Survey |
| NGA | Next Generation Attenuation |
| NGVD29 | National Geodetic Vertical Datum of 1929 |
| NEMDC | Natomas East Main Drainage Canal |
| NLD | National Levee Database |
| NLIP | Natomas Levee Improvement Project |
| NPACR | Natomas Post Authorization Change Report |
| NRCS | National Resources Conservation Service |
| NSF | National Science Foundation |
| NULE | Nonurban Levee Evaluations |
| O&M | Operations and Maintenance (USACE) |
| OCR | Over-Consolidation Ratio |
| PACR | Post Authorization Change Report |
| PDT | Project Delivery Team |
| PED | Pre-Construction Engineering and Design |
| PGA | Peak Ground Acceleration |
| PGCC | Pleasant Grove Creek Canal |
| PGL | Policy Guidance Letter |
| PNL | Port North Levee |
| Pr(f) | Probability of Failure |
| PSHA | Probabilistic Seismic Hazard Analysis |
| PSL | Port South Levee |
| P1GDR | Phase 1 Geotechnical Data Report |
| P1GER | Phase 1 Geotechnical Engineering Report |
| QA | Quality Assurance |
| QC | Quality Control |
| RD | Reclamation District |
| RM | River Mile |
| SAFCA | Sacramento Area Flood Control Agency |
| SB | Soil-Bentonite |
| SBSL | Sacramento Bypass South Levee |
| SCB | Soil-Cement-Bentonite |
| SCL | South Cross Levee |
| SGDR | Supplemental Geotechnical Data Report |

| | |
|---|---|
| SOP | Standard Operating Procedure |
| SPT | Standard Penetration Test |
| SRBPP | Sacramento River Bank Protection Project |
| SRFCP | Sacramento River Flood Control Project |
| SRN | Sacramento River North |
| SRS | Sacramento River South |
| SRWL | Sacramento River West Levee |
| SSURGO | Soil Survey Geographic Database |
| SUALRP | Sacramento Urban Area Levee Reconstruction Project |
| SWIF | System-Wide Improvement Framework Policy |
| TEC | Topographic Engineering Center |
| TM | Technical Memorandum |
| TRM | Technical Review Memorandum |
| ULE | Urban Levee Evaluations |
| USACE | U.S. Army Corps Of Engineers |
| USDA | United States Department of Agriculture |
| USGS | United States Geological Society |
| V:H | Vertical To Horizontal Ratio |
| $\overline{V}_{S30}$ | Velocity Of The Upper 30 Meters |
| VVR | Vegetation Variance Request |
| WRDA | Water Resources Development Act |
| WSAFCA | West Sacramento Area Flood Control Agency |
| WSE | water surface elevation |
| YB | Yolo Bypass |
| YBEL | Yolo Bypass East Levee |

## 1.0 INTRODUCTION

This report is an appendix to a General Reevaluation Report (GRR) for the West Sacramento Project. The project area includes portions of the Sacramento and American River Watersheds. The Sacramento and American Rivers, in the Sacramento area, form a flood plain covering roughly 110,000 acres at their confluence. The flood plain includes the City of West Sacramento, within Yolo County, California. The study area also includes other flood facilities, including the Fremont and Sacramento Weirs, Sacramento Bypass, and Yolo Bypass.

### 1.1 PURPOSE AND SCOPE

This Report presents the results of geotechnical analyses and feasibility level geotechnical recommendations to address levee height, geometry, erosion, access, vegetation, seepage, and slope stability deficiencies within the West Sacramento GRR study area. For this geotechnical engineering evaluation of the West Sacramento study area, the following tasks were performed and are summarized in this Report.

- Review currently available geology, geomorphology, and geotechnical information
- Review past performance and flood control system construction history/improvements
- Identification of levee performance deficiencies through analyses of the past performances, geotechnical analysis and engineering judgment
- Probabilistic geotechnical analysis and development of levee performance curves
- Deterministic geotechnical analysis of improvement measures and alternatives
- Erosion study of the Sacramento and American Rivers
- Seismic study of existing levees
- Development of geotechnical conclusions and recommendations

### 1.2 PROJECT DESCRIPTION

The West Sacramento Project authorization was provided in Section 209 of the Flood Control Act of 1962 (Public Law 87-874). Additional authority was provided in Section 101(4) of the Water Resource Development Act (WRDA) of 1992 (Public Law 102-580) and revised and supplemented through the Energy and Water Development and Appropriations Act of 1999 (Public Law 105-245) and 2010 (Public Law 111-85).

The following briefly outlines pertinent geotechnical information regarding a General Reevaluation Report (GRR) for the West Sacramento Project. This Report presents the results of geotechnical analyses and feasibility level geotechnical design recommendations to address levee height, geometry, erosion, access, vegetation, seepage, and slope stability deficiencies within the West Sacramento GRR study area.

The project area includes portions of the Sacramento and American River Watersheds. The flood plain includes the City of Sacramento within Yolo County, California. The study area also includes other flood facilities, including the Fremont and Sacramento Weirs, Sacramento Bypass, and Yolo Bypass. The West Sacramento GRR study area has been divided into two sub-basins; the North Sub-Basin and the South Sub-Basin, which were further subdivided into study reaches. The North Sub-Basin includes:

- 5.5 miles of the Sacramento River West (Right) Bank Levee from the Sacramento Bypass south to the confluence of the Barge Canal and the Sacramento River.
- 1.1 miles of the Sacramento Bypass South (Left) Bank Levee from the Sacramento Weir west to the Yolo Bypass Levee. 1.7 miles of the North Levee (Right) of the Sacramento Bypass levee, while not providing direct flood protection to the North Sub-basin, will be discussed to provide clarification to potential bypass widening alternatives
- 3.7 miles of the Yolo Bypass East (Left) Bank Levee from the confluence of the Sacramento Bypass and the Yolo Bypass south to the Deep Water Ship Channel Navigation Levee.
- 4.9 miles of the DWSC West (Right) Bank Navigation Levee (referred to as the Port North Levee) from the Stone Locks west to the cut in the Yolo Bypass East Bank Levee.

The South Sub-Basin includes:

- 4.0 miles of the DWSC East (Left) Bank Navigation Levee (referred to as the Port South Levee) from the Stone locks west past to the beginning of the Yolo Bypass East Bank Levee.
- 21.4 miles of the DWSC West (Right) Bank Navigation Levee from the intersection of Port North Levee and Yolo Bypass Levee south to Miners Slough. The DWSC West Bank Levee would act as the line of protection if the DWSC East Bank Levee were to breach; thus the embankment is included in the South Sub-Basin.
- 2.8 miles of the Yolo Bypass East (Left) Bank Levee from the end of Port South Levee south to South Cross Levee.
- 5.9 Miles of the Sacramento River West (Right) Bank Levee from the confluence of the Barge Canal and the Sacramento River south to the South Cross Levee.
- 1.2 Miles of the Babel Slough North Levee (referred to as the South Cross Levee) DWSC to the Sacramento River.

The West Sacramento GRR is evaluating federal interest in alternatives to reduce flood risk in the study area. The West Sacramento GRR has identified several technical deficiencies associated with the flood risk management system protecting the study area. There are various alternatives under consideration to address these deficiencies and the geotechnical components of those alternatives are discussed and or evaluated in this report. The alternatives consist of a

combination of structural measures to mitigate potential seepage and slope stability distress, erosion protection, and evaluate a closure structure on the Deep Water Ship Channel (DWSC) as a constructible element in conjunction with proportionate structural measures for seepage and stability mitigation.
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## 1.3 PROJECT STATIONING

In this report, project stationing (Sta. XX+XX) is the primary method used to describe locations. However, several various alignments have been developed which may occasionally be referenced including the Department of Water Resources (DWR) Urban Levee Evaluation (ULE) stationing, levee mile (LM), river mile (RM), and USACE O&M Levee Unit. Table 1-1 shows the analysis sections within the study area of the West Sacramento Project, in terms of RM and LM and maintenance agency where applicable.

**Table 1-1: West Sacramento GRR Project Levees**

| Basin | Analysis Section | Maintenance Agency[1] | Unit | LM | RM |
|---|---|---|---|---|---|
| NORTH | PNL-STA. 117+37 | Port of West Sacramento | - | - | 42.83 |
| | SBSL-STA. 32+00 | DWR-MA08 | 2 | 0.62 | 1.22 |
| | SBSL-STA. 52+00 | | | 0.24 | 1.60 |
| | SRWL-STA. 96+00 | DWR-MA04 | 1 | 1.2 | 61.67 |
| | SRWL-STA. 190+00 | | | 2.59 | 30.20 |
| | YBEL-STA. 36+00 | RD 900 | 2 | 1.89 | 41.90 |
| | SBNL-STA. 8+30 | DWR-MA08 | 1 | 1.29 | 0.40 |
| SOUTH | DWSCWL-STA. 12+00 | USACE | - | - | 41.21 |
| | PSL-STA. 123+55 | Port of West Sacramento | - | - | 43.45 |
| | SCL-STA. 17+50 | RD 900 | - | - | 38.25 |
| | SRWL-STA. 264+00 | RD 900 | 1 | 2.80 | 53.74 |
| | SRWL-STA. 80+00 | | | 6.33 | 53.08 |
| | SRWL-STA. 35+22 | RD 765 | 1 | 0.67 | 51.07 |
| | YBEL-STA. 10+00 | RD 900 | 2 | 3.24 | 40.82 |
| | YBEL-STA. 53+96 | RD 999 | 1 | 1.07 | 37.22 |

Note – MA: Maintenance Area, RD: Reclamation District

## 1.4 PROJECT DATUM

Elevation references in this report are in feet and are based on the North American Vertical Datum of 1988 (NAVD88) unless otherwise noted. Conversion factors ranged between +2.44 to +2.54 feet were obtained from the software program Corpscon 6.0, produced by the USACE Topographic Engineering Center (TEC), Survey Engineering and Mapping Center of Expertise, was applied to convert Geodetic Vertical Datum of 1929 (NGVD29) elevations to NAVD88. All horizontal references in this report are in feet and are based on the California State Plane, Zone II, North American Datum of 1983 (NAD83).

## 1.5 SOURCES OF DATA

The subsurface conditions and material properties of the levee embankment and foundation soils have been characterized by several studies in the past. These studies have been prepared for feasibility and design projects by the USACE, DWR, and WSAFCA among others.

Through Assembly Bill AB 142, the State has appropriated $500 million of funding to DWR to begin a comprehensive program of levee evaluation and upgrades. The ULE Program evaluates levee systems estimated to protect more than 10,000 people. DWR has retained a team led by URS Corporation (URS) to assist in the geotechnical evaluation of the state's project levees. The ULE Program has generated Technical Review Memorandums (TRM), Phase 1 Geotechnical Data Reports (P1GDR), Supplemental Geotechnical Data Reports (SGDR), Phase 1 Geotechnical Evaluation Reports (P1GER), and Geotechnical Evaluation Reports (GER) for the Study Area.

The available geotechnical data from the above mentioned sources includes borings and CPTs drilled along the levee; crest, landside toe and field, and waterside toe, geology and geomorphology studies, and geophysical surveys. The levee geometry was based on the existing data in the National Levee Database (NLD) supplemented by recent Light Detection and Ranging (LiDAR) survey and bathymetric survey provided by the DWR as part of the ULE program. A summary of reference documentation is contained in Section 18.0

## 1.6 WITHOUT PROJECT CONDITIONS DESCRIPTION

Levee construction and remediation has occurred within the study area since the middle of the 19th century. While the modern levee system was constructed in the early 20th century and remediated in the 1940s through 1950's, the vast majority of the construction and remediation consisted of soil embankment alterations through various methods. Beginning in the early 1990s and continuing through present day, internal improvements have been and continue to be constructed. These mostly consist of through and underseepage cutoff walls as well as placement of a stability berm and related features to address through seepage. The following paragraphs present how the modern improvements have been incorporated in the West Sacramento project and details the without project conditions.

In coordination between USACE, WSAFCA, the Reclamation Board, and the DWR two flood control project have been completed. The first, constructed from 1990 to 1993, as part of the Sacramento Urban Area Levee Reconstruction Project (SUALRP). Under SUALRP, a stability berm and related features to address through seepage along the entire length of the Sacramento River levee bordering the Southport area were constructed. The second, the West Sacramento Project, constructed levee raises on portions of the southern levee of the Sacramento and Yolo Bypass between 1998 and 2002 to provide the City of West Sacramento with greater than 200yr level protection.

When the design efforts of the West Sacramento Project neared completion, underseepage was noted along the RD 537 maintained portion of Sacramento Bypass south levee in 1997. Downstream of RD 537, the Yolo Bypass east levee, which is adjoining to the Sacramento Bypass south levee and maintained by RD 900, experienced stability issues in 1998 along the levee in 1998. The City of West Sacramento, RD 537 and RD 900 requested the USACE to conduct further geotechnical investigations and incorporate design changes to mitigate these issues. The completed West Sacramento Project included the incorporation of the entire reconstruction of one section of RD 537 levee replacing the original clay and organic material within the embankment and upper foundation with engineered fill and construction of a 60-70ft deep slurry wall to mitigate under seepage at the confluence of the Sacramento and Yolo Bypass (RD 900).

## 1.7 WITH PROJECT CONDITIONS DESCRIPTION

The West Sacramento GRR is evaluating federal interest in alternatives to reduce flood risk in the study area. The West Sacramento GRR has identified several technical deficiencies associated with the flood risk management system protecting the study area. There are various alternatives under consideration to address these deficiencies and the geotechnical components of those alternatives are discussed and or evaluated in this report. The alternatives consist of a combination of structural measures to mitigate seepage and slope stability, provide erosion protection and include non structural measures such as widening of the Sacramento Bypass to lower the risk. The with project conditions will address project authorization covering a range of levels of protection. Notably, the range is bounded from a 25yr to 500yr level of protection. Typically, the with project condition will achieve a 200yr level of protection. In certain locations it should be noted that the existing levee height may be at an elevation above the 200yr requirement and range to approximately meet a 500yr requirement.

## 2.0 GEOLOGY AND GEOMORPHOLOGY

### 2.1 GEOLOGIC SETTING

The West Sacramento GRR study area lies in the central portion of the Sacramento Valley which lies in the northern portion of the Great Valley Geomorphic Province of California. The Sacramento Valley lies between the northern Coast Ranges to the west and the northern Sierra Nevada to the east, and has been a depositional basin throughout most of the late Mesozoic and Cenozoic time. A large accumulation of sediments, estimated over two vertical miles in thickness in the Sacramento area, were deposited during cyclic transgressions and regressions of a shallow sea that once inundated the valley. This thick sequence of clastic sedimentary rock units was derived from adjoining easterly highlands erosion during the Late Jurassic period with interspersed Tertiary volcanics. They form bedrock units now buried in mid-basin valley areas. These bedrock units were covered by coalescing alluvial fans during Pliocene-Pleistocene periods by major ancestral west-flowing Sacramento Valley rivers (Feather, Yuba, Bear, and American). These rivers funneled large volumes of sediment into the Sacramento basin. Late Pleistocene and Holocene (Recent) alluvial deposits now cover low-lying areas. These deposits consist largely of reworked fan and stream materials deposited by meandering rivers prior to construction of existing flood control systems. Figure 2-1 shows the surficial soil deposits of the Sacramento region based on a reconnaissance soil survey performed by the United States Department of Agriculture (USDA) in 1913.

The Sacramento River is the main drainage feature of the region flowing generally southward from the Klamath Mountains to its discharge point into the Suisun Bay in the San Francisco Bay area. Located in central northern California, the Sacramento River is the largest river system and basin in the state. The 27,000 square mile Sacramento River Basin includes the eastern slopes of the Coast Ranges, Mount Shasta, and the western slopes of the southernmost region of the Cascades and the northern portion of the Sierra Nevada. The Sacramento River, stretching from the Oregon border to the Bay-Delta, carries 31% of the state’s total runoff water. Primary tributaries to the Sacramento River include the Pit, McCloud, Feather, and American Rivers. Within the Sacramento area, the Sacramento and American Rivers have been confined by man-made levees since the turn of the century. The confluence with the Sacramento River, only 20 feet above sea level, is subject to tidal fluctuation although more than 100 miles north of the Golden Gate and San Francisco Bay. Within the study area, these levees were generally constructed on Holocene age alluvial and fluvial sediments deposited by the current and historical Sacramento River and its tributaries. Pleistocene deposits underlie the Holocene deposits. The Sacramento River Basin and associated subregions are shown on Figure 2-2.

The study area has been mapped by a number of geologists on a regional scale including published maps by Jennings et al., (1977), Wagner et al., (1981), and Helley and Harwood (1985). The Jennings and Wagner maps are both compilation maps that reflect mapping by previous authors and thus show geologic interpretation similar to those of Helley and Harwood. Helley and Harwood’s mapping focused on Quaternary geologic units based on geomorphology and was performed at a scale of 1:62,500.

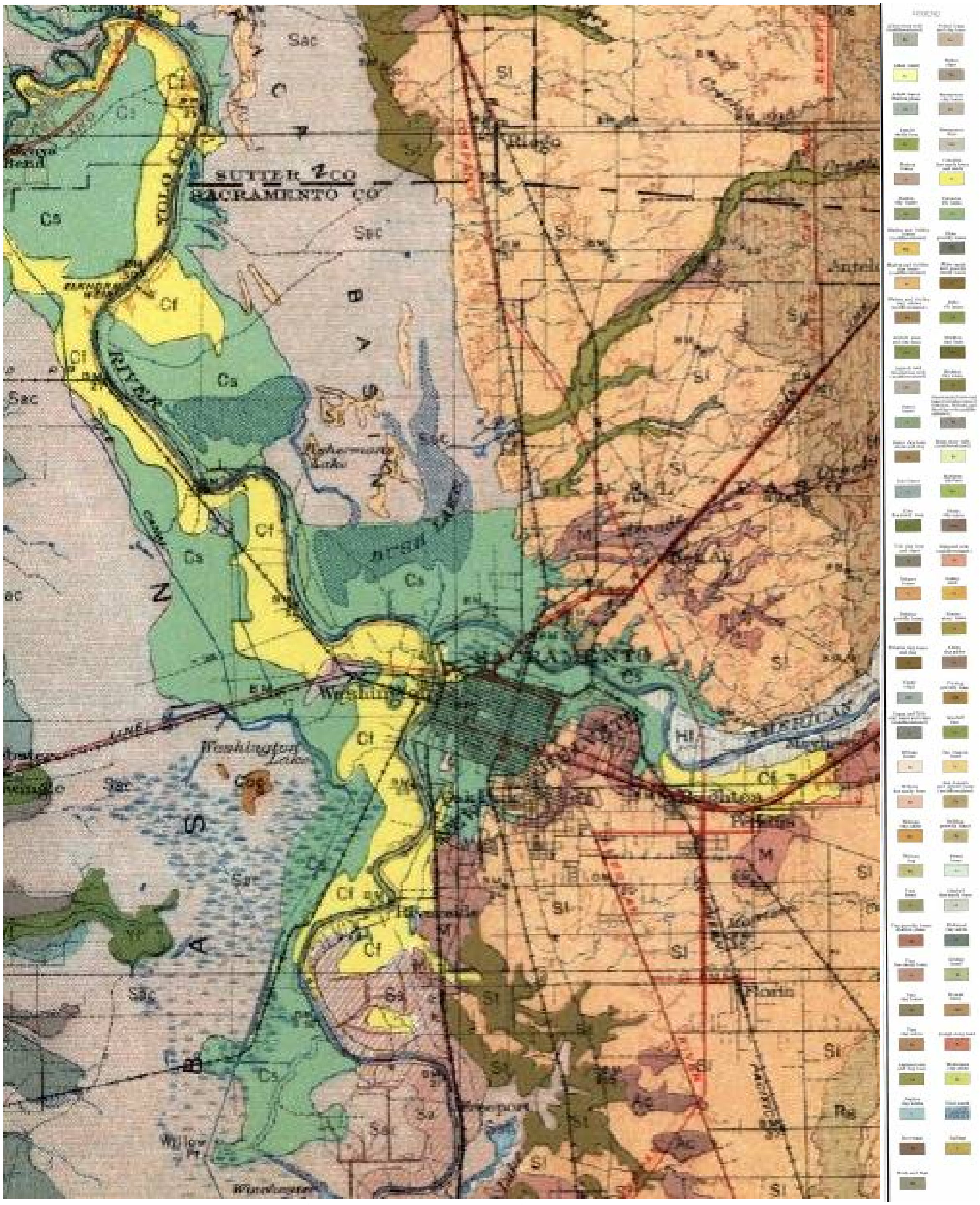



**Figure 2-1: USDA Surficial Soil Survey of the Sacramento Region, 1913**


**Figure 2-2: Map of the Sacramento River Basin**

## 2.2 GEOMORPHOLOGY

Prior to the late Pleistocene (10,000 to 30,000 years ago), the Sacramento River Basin depositional environment was influenced by a lowered base level due to sea levels as low as 400 feet below present (Harden 1998). These lowered global sea levels would have had their greatest influence in present coastal areas such as the San Francisco Bay area, but based on interpretation of the depth to denser, coarser Pleistocene soils it is estimated that average river levels in this area could have been 50 to 60 feet below current levels. The rivers would have been characterized by high energy flow with greater downward erosion rather than deposition, and would have had greater capacity to carry and deposit sand and gravel deposits into the project area. This older geomorphology is largely covered by the more recent (Holocene) sediments in the project area. The thick zone of materials deposited above the dense, older Pleistocene alluvial deposits are therefore less than 10,000 to 30,000 years old, which is reflected in these deposits consisting of very soft to firm clays and silts and abundant loose to medium dense sands.

The filling of the Sacramento Valley with sediments following the rise in sea level to the current level has significantly reduced the gradient of the rivers flowing down from the Sierra Nevada and Klamath Mountains (including the Sacramento and American Rivers). This gradient reduction has caused the energy of these rivers to transition from erosional to graded. Graded rivers are characterized by downward erosion that is less dominant and more directed toward side-to-side movements than down-cutting. The lateral energy of a graded river causes synchronous erosion and deposition in sweeping bands commonly referred to as meanders. The outside of the meander is a zone of erosion. Material removed by the river at this zone is then deposited downstream as point bars in zones of decreased velocity on the inside of the subsequent meanders. In this way, the river migrates laterally across the flood plain. Often this erosion is slowed where the river encounters more resistant materials in the flood plain. This allows the next closest upstream meander to catch up and gradually erode away the “neck” between the two meanders. Flooding often accelerates this process as the higher energy flows can more easily cut a new thalweg (base of the active channel). The result of the conjoining meanders is the straightening of the river across the opening of the neck and the creation of an abandoned bend in the river, commonly referred to as an oxbow lake.

Because of the low topographic position and proximity to the confluence of the two large rivers, the West Sacramento area has been subjected to periodic inundation by floodwaters during late Holocene time, and consequently is underlain by a relative thick package of young alluvial deposits. The floodwaters of the Sacramento River deposit fine sand and silt-rich alluvium along the flanks of the river bank, and carry finer-grained clay and silt in suspension onto the distal floodplain. This sorting process creates a “natural levee” landform with a topographic gradient that slopes away from the river. The topographically low area west of the Sacramento River, known as the Yolo Basin, was a frequently inundated swampland prior to historic reclamation. Flood overflow fed thousands acres of sloughs, swamps, and dense marshes of bulrushes creating a region then known as the Tule, and today as the Yolo Basin. Sources of water and sediment contributing to the Yolo Basin include not only the Sacramento River, but the Cache Creek and Putah Creek systems directly northwest and west of West Sacramento, respectively. Cache and Putah Creek channels do not currently connect directly to the Sacramento River, and deposit clay, silt, and fine sand into the low-lying area of Yolo Basin via a network of sloughs, channels, small sinks (lakes) and islands.

## 2.3 HYDRAULIC MINING

Hydraulic mining activity in the Sierra Nevada during the mid- to late-1800s supplied a substantial amount of sediment to many river channels draining the Sierra Nevada, which resulted in aggradation of the channels and flooding due to decrease in channel cross section area. Gold dredging and mining operations have destroyed some fluvial deposits and surfaces, confounding the understanding of the long-term geomorphic history.

This phenomenon, coupled with a disastrous flood in 1862, prompted the channelization of the Sacramento and American Rivers and re-alignment of the American River to its present-day configuration, from the former confluence with the Sacramento River to about two miles upstream. It was hoped that these actions would provide flood control as well as stimulate the flushing of accumulated mining-derived sediment from the channel.

## 2.4 SACRAMENTO BYPASS AND SOUTH CROSS LEVEE GEOLOGY

The Sacramento Bypass levee and South Cross levee at Garcia Bend traverse the study area in a generally east-west orientation and thus overlie coarser-grained river deposits and finer-grained basin deposits, from east to west. Shallow subsurface deposits here should interfinger and alternate between the river and basin deposits, reflecting changes in the position of river and basin depositional processes in time. Also, because these two levees are sub-orthogonal and proximal to the present-day river, there may be complex erosional relationships in the subsurface stratigraphy from past positions of the Sacramento River.

The stratigraphic deposits beneath the Sacramento Bypass levee vary from east to west and vertically with depth. The deposits directly beneath the levee consist of Holocene and historical splay and overbank deposits from the Sacramento River, laid down prior to the construction of the Sacramento Bypass levee, and chiefly consist of soft to medium stiff silt and clay with sand in the upper 10 feet. The sediment has more silt and sand closer to the river, grading to silt and clay westerly. At the surface, a historical crevasse splay deposit is delineated beneath the Sacramento Bypass levee in this reach, extending toward the northwest. The splay is well-expressed in aerial photographs, and trends “up valley” following the slope of the natural levee toward the basin. The levee fill overlies this feature. The crevasse splay deposit is a locally sandier deposit about two- to three-feet-thick, mantling the adjacent sediment. About 20 feet of Holocene sandy silty clay and fat basin clay underlie much of the historical alluvium beneath the levee. Two layers of sand and gravel in turn underlie the Holocene alluvium and basin deposits, each about 20 feet thick. These layers are encountered deeper in the subsurface environment along and are separated by a hard sandy silt to silt. Adjacent to the Sacramento River, the coarse grained deposits are not present in the borings which show soft, fine-grained deposits consisting of chiefly elastic silt. These soft, fine-grained sediments may have been deposited in former flood-basin, lagoonal, or abandoned-channel environments. Deeper subsurface gravels, perhaps representing high-energy Pleistocene floodplain deposits, may extend north-south beneath the levee.

The South Cross levee connects the Sacramento East Levee River  with the Yolo Bypass East Levee and crosses the transition between coarser-grained natural levee deposits (Holocene Alluvium, Ha) and finer-grained basin deposits (Qs) primarily consisting of medium stiff fat clays and elastic silts.  High plasticity fat clay is present at the ground surface and along the western half of the reach which coincides with the characteristics of marsh deposits.  Deeper foundation deposits include medium dense to dense silty sands with increasing clay trending westward.

## 2.5 SACRAMENTO RIVER GEOLOGY

Along the eastern side of the study area, adjacent to the Sacramento River, the subsurface stratigraphy is complex. The stratigraphy is expressed as abrupt lateral or vertical changes in sediment grain size and/or consolidation. This pattern is a result of the dynamic processes commonly associated with large rivers, such as: (1) post-depositional erosion of sediments and subsequent backfilling with different sediments; (2) river migration and resulting meander scroll deposits (Figure 2-3); or (3) local crevasse splay and overbank activity. Generally, the subsurface stratigraphy adjacent to the river exhibits a fining-upward sequence of gravel, sand, silt. Gravel of about 20 feet thick appears laterally extensive at the base of the aquifer layer in the northern part of the map area, and underlies both sides of the Sacramento River near the I Street Bridge; whereas, in the south part of the map area (i.e., downstream of the Deep Water Ship Channel), gravel is only locally present or is absent.

The Sacramento River has irregular sinuosity south of the confluence with the American River, with both large and small radius-of curvature meander bends. The river has, in places, laterally migrated over the past thousands of years, with erosion occurring on the outsides of bends, and deposition of younger sand-rich sediment occurring on the insides of the river bends. Geologically older and erosion-resistant Riverbank Formation is present at the ground surface south and east of the city of Sacramento, and younger alluvium is inset into this formation. Additionally, because of the low topographic position and proximity to the confluence of the two large rivers, the Sacramento area has been subjected to repeated inundation by floodwaters during the past several thousand years. The floodwaters deposit fine sand and silt-rich alluvium along the flanks of the river bank and finer-grained clay and silt are carried in suspension onto the distal floodplain. This hydraulic sorting process creates a ‘natural levee’ landform with a topographic gradient that slopes away from the river. Consequently, the levee is underlain by a variable, relatively thick, and relatively young, sandy and silty, unconsolidated alluvial deposits.

South of the confluence of the American River, the Sacramento River demonstrates a complex relationship of fluvial deposits at the surface and beneath the eastern floodplain of the Sacramento River. The surface and subsurface distributions of sandy and clayey deposits are a function of former river positions on the landscape, and present-day geomorphic processes adjacent to the river channel. The levees are underlain entirely by geologically young, unconsolidated, silty and sandy fluvial deposits.

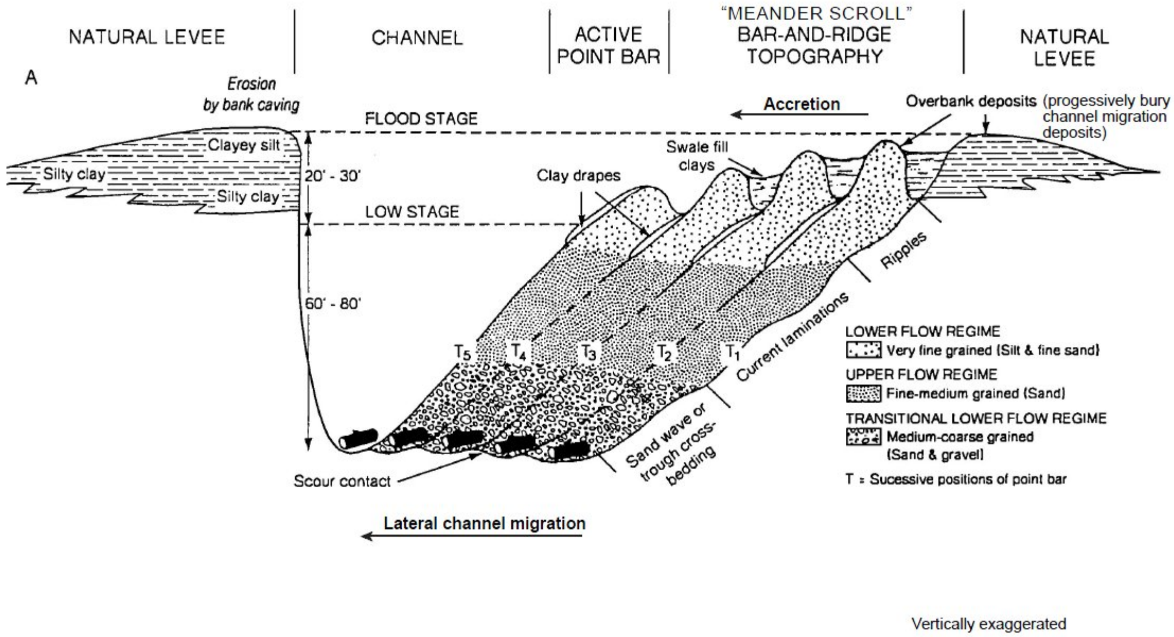



**Figure 2-3: Cross Section of a Meander Scroll**

## 2.6 YOLO BYPASS GEOLOGY

Broadly speaking, west of the present-day Sacramento River, relatively thick packages of elastic (fat) clay comprise the upper stratigraphy of the marsh and basin deposits. The basin deposits typically are up to about 20 feet thick, and in rare instances, up to 80 feet thick, and occasionally are interbedded with soft-to-stiff silt or medium dense sand and silty sand. Packages of dense coarse-grained (i.e. sand and gravel) deposits generally occur below present-day sea level, and probably represent latest Pleistocene deposits now buried by Holocene basin deposits.

## 2.7 PORT NORTH AND PORT SOUTH LEVEE GEOLOGY

The present-day Port North and Port South region is generally comprised of fine-grained silt and clay and fine sand basin depostits (Qn) of the Holocene period which primarily trend westward. The basin deposits may be obscured by cultivation from agricultural activities in the region. The Port South Levee is intersected from the south by a marsh deposit which trends in a north-south manner containing organically rich silts and clays. The channel within the levee embankments is predominantly laden with open active stream channel without permanent vegetation.

## 2.8 DEEP WATER SHIP CHANNEL EAST AND WEST LEVEE GEOLOGY

The Deep Water Ship Channel East and West Levees regionally overlies, moving from south to north numerous distinct units which include: remnant islands (knobs) of a Pleistocene alluvial fan that may be derived from the Putah Creek unit (Pf – semi-consolidated silts, sands, sandy clays and fine to coarse subrounded gravel), marsh deposits (Qs – silts and clays likely rich in organics) and basin deposits (Qn – fine sands, silts and clays subject to recent cultivation). Existing subsurface data suggests that the Pleistocene fan areas are medium dense to dense sand with silt at roughly twenty feel below the levee base and are covered by elastic clays. Surficial deposits along the Deep Water Ship Channel are fine-grained and stiff to very stiff, and may have lessened susceptibility to underseepage relative to the Sacramento River due to the overall low permeability characteristics of the thick basin deposits.

## 3.0 CONSTRUCTION HISTORY

A mix of Federal, State, and local agencies have been involved in flood control project construction and operation since levees were first constructed in California in the mid-1800's. Since the creation of the State Reclamation Board (now the Central Valley Flood Protection Board or CVFPB) in 1911 and the authorization of the Sacramento River Flood Control Project (SRFCP) in 1917, most levee improvements have been first Federally authorized by Congress, then subsequently authorized by the State Legislature.

The SRFCP was authorized by the Flood Control Act of 1917 (PL 64-367) as modified by the Acts of 1928, 1937, 1941 and 1950. Features of the SRFCP, in the study area, consisted of levees along the Sacramento and Yolo Bypass and the Sacramento River, including new and reconstructed levees. The completed flood control system was documented in 1957 in a design memorandum, which included design water surface profiles. To this day, these are the profiles that govern the operation and maintenance requirements of the levee system.

### 3.1 SACRAMENTO AND YOLO BYPASS LEVEES

In 1927, the California State Legislature specified the portions of the SRFCP that would be operated and maintained by the State of California; the Sacramento and Yolo Bypasses was included as two of these features. The construction method of the Sacramento Bypass levees is not known; however, it was built as part of the SRFCP and likely using the same method as the Yolo Bypass levees. The Yolo Bypass levees were constructed using the clamshell method where a clamshell was used to excavate material from the waterside toe of the levee and then pile the material to form the levee. After the excavated material consolidated, the levees were dressed and shaped to their final form. This construction method usually resulted in a ditch at the waterside levee toe. Figure 3-1 shows the dredge Vulcan constructing levees on the Yolo Bypass just south of West Sacramento around 1911. There was typically no compaction of the material placed for levees constructed with this method. Therefore, the material in the levee is usually loose and consisting of materials similar in composition to the surrounding native materials; primarily silts, clays and fine sands typical of basin deposits as well as, on portions of the Sacramento Bypass, which contain coarse sands with minor gravel lenses typically noted in splay deposits.

The West Sacramento Project was authorized in the WRDA of 1992 and the design was documented in the 1996 Basis of Design report. The West Sacramento Project consisted of raising and enlarging several levee sections of the Sacramento and Yolo Bypass. Contract A was completed in 1998 and consisted of levee raises, widening, berms, and internal drainage systems on the Yolo Bypass levee from the DWSC to the Sacramento Bypass, Figure 3-2. Contract B was completed in 1999 and consisted of levee raises, widening, berms, internal drainage systems, and a waterside cutoff wall, Figure 3-3. Repairs due to flood events to the Contract A levees were completed in 2010 and 2011 as part of Contract C and D respectively which included a stability berm, internal drainage systems, slope flattening and levee widening. The WSAFCA constructed a soil-bentonite cutoff wall along the levee centerline through portions of the Contract B reach as part of their CHP Academy Early Implementation Project in 2011 as a response to seepage deficiencies during the 2006 flood event.

**Figure 3-1: Dredge Vulcan Constructing Yolo Bypass Levee South of West Sacramento**

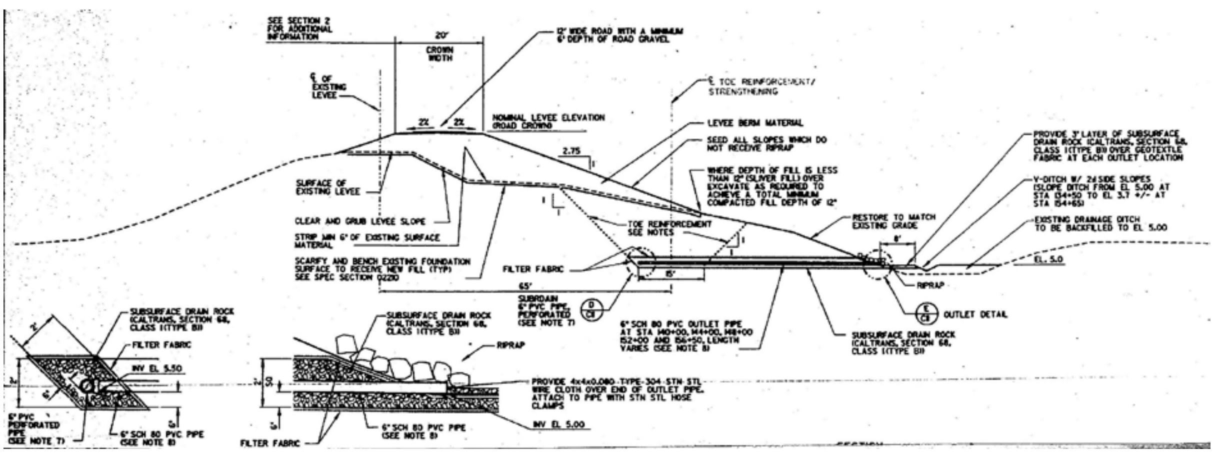



**Figure 3-2: West Sacramento Project Contract A Typical Section**

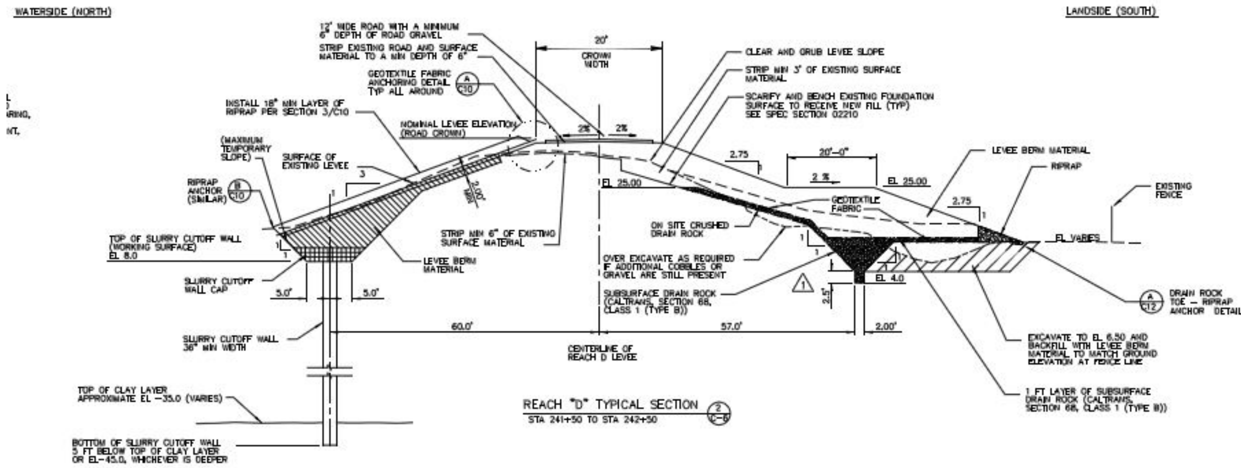



**Figure 3-3: West Sacramento Project Contract B Typical Section**

## 3.2 SACRAMENTO RIVER WEST LEVEE

The levees along the Sacramento River were constructed by local interests using clamshell dredges excavating material from the Sacramento River in the early 1900's. Figure 3-4 shows the Dredge Neptune placing material at RM 57.3 in 1942 during construction of the Sacramento Bank Protection Project. Figure 3-5 was taken around 1911 near Davis Road in West Sacramento and shows the recently constructed Sacramento River levee. This method of construction usually resulted in loose, sandy fill material that is deepest below the center of the levee. The current materials within the levee embankment are predominantly sands, silty sands, and cohesionless materials mainly silts and gravels. Numerous riverbank and levee waterside slope protection were constructed along the Sacramento west bank levee.

In 1990 the SUALRP constructed a drained stability berm along the Sacramento River levee from the DWSC to the South Cross levee, a typical section is shown in Figure 3-6. The WSAFCA constructed a DSM cutoff wall (approximately 130ft in depth) and a shallow soil-bentonite cutoff wall (approximately 35ft in depth) as part of the Rivers and I Street EIPs in 2011 and 2010 respectively.  The Rivers EIP DSM wall provided mitigation for underseepage while conversely the I Street EIP shallow wall mitigated for through seepage concerns.

**Figure 3-4: Dredge Neptune at RM 57.3 in 1942**


**Figure 3-5: Levee Constructed Near Davis Road, West Sacramento**

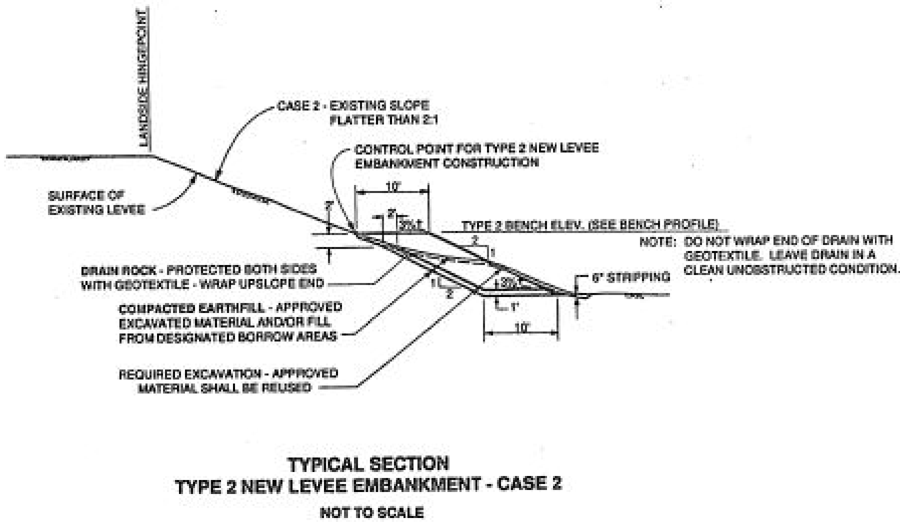



**Figure 3-6: Sacramento River Levee - Typical Stability Berm Section**

## 3.3 DEEP WATER SHIP CHANNEL, PORT NORTH AND SOUTH, AND NAVIGATION LEVEES

In late 1940s through the 1960s the USACE designed and constructed a navigation levee east of the Yolo Bypass levee, the DWSC was constructed via dredging operations west of this levee to allow ship traffic into the Port of West Sacramento. The DWSC cut through the project levee and a new navigation levee constructed west of the DWSC to separate the DWSC from the Yolo Bypass. The construction methods are not known but likely using clam shell using materials from the excavation of the channel. The levee embankments are comprised of predominantly silts, clays, and fine sands typical of marsh and basin deposits respectively.

## 3.4 SOUTH CROSS LEVEE

No construction history was available regarding the south cross levee as it is a non-federally constructed, operated, and maintained levee. The levee embankment typically contains high plasticity (fat) clays and silts.

## 4.0 PAST PERFORMANCE

Despite levee improvements, recent flood events in 2006, 1997, 1986, and 1957 have caused levee distress in the form of seepage, boils, and slope instability. The levee embankments were approximately loaded 30% to 50% of the effective levee height during these events.

Erosion events were noted on the Sacramento Bypass South levee, Yolo Bypass East levee, and the Sacramento River West levee during the events of 1997 and 2006. These events, most prevalent on the Yolo Bypass East levee and less so on the Sacramento River West levee, can be attributed to high water, wavewash, surface runoff, pier scour adjacent to bridge abutments, or movement of rock revetment.

### 4.1 SACRAMENTO RIVER BYPASS SOUTH LEVEE

During the high water events of 1997 and 1998, multiple seepage boils occurred along the Sacramento River Bypass Levee just landward of the levee toe in between RM 0.6 and RM 1.7 which required floodfighting. The seepage boils ranged in diameter from 2 to 12 inches in diameter and were ringed with sandbags as a floodfighting measure. The embankment was loaded to approximately 50% of the levee height for the flood events of 1997 and 1998. Underseepage was found extending into the CHP Academy according to CHP personnel, but DWR personnel indicated that the drainage originated from the drain beneath the seepage berm.

### 4.2 SACRAMENTO RIVER WEST LEVEE – NORTH BASIN

In April 2006, a segment of the Sacramento North Levee along Fountain Drive (west of Westlake Drive) experienced heavy seepage and boils along the landside toe according to eyewitness reports. Water was seen bubbling up around a large fence pillar and from a buried irrigation control box in an area recently developed for residential use. The water surface elevation at that time was 29.8 feet (NGVD29) , 32.3 feet (NAVD88), at the I-Street Bridge staff gage. Also along this levee, 470 lineal feet of sloughing on the waterside embankment just south of the Tower Bridge was reported during the 1997 flood event. The sloughing was intermittent over the 470 lineal feet, and ranged dimensionally from 4-16ft in width to 2-10ft in depth.

### 4.3 SACRAMENTO RIVER WEST LEVEE – SOUTH BASIN

Many seepage and slope stability problems arose along the Sacramento River South levee during the flood events of 1997 and 2006. In 1997, numerous slides and sloughing occurred on the waterside embankment between RM 57.5 and RM 56.5. Dimensionally, the sloughs ranged from 4-8ft vertical faces and instability ranged in length from 100 feet to over 700 feet potentially induced by an erosion event. Further downstream, in the area of Bee’s Lakes, pin boils were observed along the landside toe of the secondary levee. Finally, in the region extending from Oak Hall Bend to Clay Bank Bend, three slides occurred that were up to 300 feet in length with 3-5ft vertical faces. In 2006, between Chicory Bend (RM 55) and Oak Hall Bend (RM 54), numerous seepage boils were reported near the landside toe near Davis Road.

## 4.4 YOLO BYPASS EAST LEVEE

In 1998, approximately one half-mile south of Interstate 80, the excavation of an exploration trench along the landside toe produced significant fissures and cracks indicating the initiation of a slide along a portion of the levee. In the same area in 2006, multiple slips were observed on the waterside slope after a prolonged storm event.

In the region just north of Interstate 80, three slides were observed on the landside embankment in 1995. The sliding started in January and continued at a slow rate until the end of March. The most prominent slide was 100 feet long and had a vertical displacement of two feet at the headscarp. The water elevation in the Yolo Bypass was 22 feet (NGVD29) at the time of the slip. In the same area in 2006, seepage was noted through and under the landside embankment which resulted in a shallow toe slide that was 75 feet in length and about 75 feet wide. Vertical displacement at the headscarp was about 1.5 feet. Finally, in the area just south of the UPRR line, two slope failures occurred on the waterside embankment in 2001, presumably due to the presence of an organic layer in the foundation.

Two landside slope failures were observed along the Yolo Bypass levee just north of the UPRR line in February 1983. The first slide had a base width of 114 feet and had a vertical displacement of 4 feet at the headscarp. The second slide had a base width of 89 feet and had 9 feet of vertical displacement at the headscarp.

These slides occurred presumably due to the presence of a weak organic layer with inadequate shear strength along with development of excess pore pressures due to underseepage and through seepage within the upper foundation and embankment.

## 4.5 DEEP WATER SHIP CHANNEL EAST LEVEE

In 2006, incidents of landside instability were reported. The instability occurred in a region just north of the South Cross levee and were generally shallow, rain-induced slumps that were considered maintenance issues.

## 4.6 PORT NORTH AND PORT SOUTH LEVEES

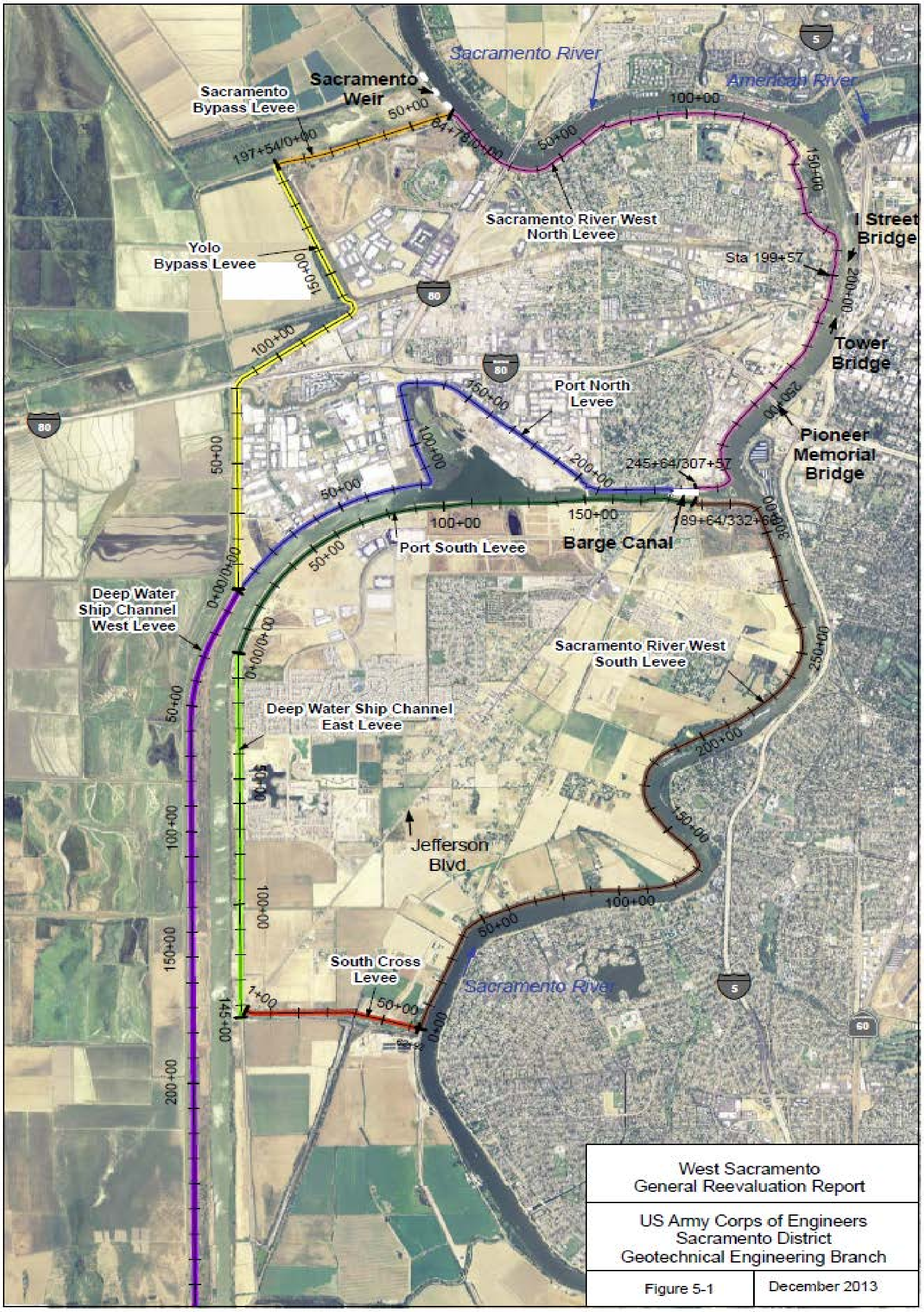
Limited information is available as to the past performance of both the Port North levee. CA DWR reported seepage distresses via field observations in numerous areas throughout the entire alignment in both 1963 and 1965. This area is located from the barge canal to the beginning of the DWSC due west of the Port of Sacramento.

## 4.7 SOUTH CROSS LEVEE

CA DWR reported seepage distresses via field observations in the extent areas of the alignment in both 1963 and 1965. These locations are noted as the eastern most portion near the Sacramento River West Levee; and western most area nearing Jefferson Blvd. and the DWSC East Levee.

## 5.0 GEOTECHNICAL REACH DESCRIPTIONS

The following sections describe the geometric project features and locations. Figure 5-1 displays the study area and project features.



## 5.1 WEST SACRAMENTO – NORTH BASIN

The North Basin of the West Sacramento Project includes levees on the south bank (left) of the Sacramento Bypass, west bank (right) of the Sacramento River from the Sacramento Bypass downstream to the Stone Lock structure and continues on the north bank of the Port of Sacramento (right) to the Yolo Bypass East Levee (left) thence upstream to meet the Sacramento Bypass south levee. Table 5-1 displays data on the levee alignment for each channel.

**Table 5-1: West Sacramento – North Basin – Levee Properties**

| Channel | Begin | End | Crest Width (ft) | LS Levee Slope | WS Levee Slope | Levee Height (ft) |
|---|---|---|---|---|---|---|
| | Sta. | Sta. | | | | |
| SBSL | 0+00 | 64+80 | 20-30 | 2.0-2.5:1 | 2.0-2.5:1 | 15-25 |
| SRWL | 0+00 | 307+60 | 20-30 | 2.0-2.5:1 | 3.5:1 | 15-25 |
| PNL | 0+00 | 245+65 | 20-25 | 2.5-3.0:1 | 2.75-3.0:1 | 5-8 |
| YBEL | 0+00 | 197+55 | 20-30 | 2.5-3.0:1 | 3.0:1 | 15-20 |
| SBNL | 0+00-DWR | 33+66-DWR | 15-20 | 3.0:1 | 2.5-3.0:1 | 15-20 |

### 5.1.1 SACRAMENTO BYPASS SOUTH LEVEE

As part of the Rivers EIP construction the maintaining agency, CA DWR removed the vegetation in compliance with current guidance. In some areas, there is moderate landside vegetation (mostly large trees) existing near the levee toe, but few at the levee toe or on the levee slope. Encroachments include utility poles near the landside toe along the levee alignment. The levee crest surface is an aggregate road base with access ramps following the alignment on the landside levee slope.

The levee embankment is predominantly comprised of poorly graded sands and silty sand at the upstream portion (Sta. 35+00 to Sta. 64+80) and more finer grained silts and fat clays nearing the downstream end (Sta. 0+00 to Sta. 35+00). The levee is underlain by a thick (15-20ft) silt and clay blanket layer which is underlain by pervious poorly graded sand and gravel aquifer.

### 5.1.2 SACRAMENTO RIVER WEST LEVEE

On the Sacramento River west levee (Sta. 0+00 to Sta. 307+60) there is significant vegetation on the waterside bench which varies in thickness. Typically within the reach, the waterside bench becomes wider moving downstream from the confluence with the Sacramento Bypass and vegetation increases to a point (Sta. 190+00) and then begins to taper in width heading towards the more downstream portions nearing the Stone Lock. In some areas, there is significant landside vegetation (mostly large trees) existing near the levee toe, or on the levee slope. On the landside numerous encroachments including fences at or near the landside levee toe, parking lots built, significant residential/commercial developments and industrial facilities nearing the downstream portion of the alignment exist. The levee crest surface varies between asphaltic concrete pavement and aggregate road base with numerous access points across the alignment within the adjacent residential/commercial developments and at the I St. Bridge, as well as near the Stone Lock structure.

The levee embankment is predominantly comprised of poorly graded silty sands, silty sands, and silts. The levee is underlain by a thin (5-10ft) silt and clay blanket layer which is underlain by pervious poorly graded sand and silty sand aquifer.

### 5.1.3 PORT NORTH LEVEE

The Port North levee (Sta. 0+00 to Sta. 245+55) contains sparse riparian habitat (vegetation) adjacent to the levee embankment. There is very little landside vegetation existing near the levee toe or on the levee slope. On the waterside bench moderate vegetation exists, mostly trees lining the turning basin of the Port of West Sacramento. Encroachments include utility poles near the landside toe along the levee alignment, multiple railroad tracks, and commercial developments. The levee crest surface is an aggregate road base with access points along the alignment within the Port of West Sacramento facility and in the adjacent commercial developments near the downstream portion of the alignment (Sta. 0+00 to Sta. 80+00).

The levee embankment is predominantly comprised of fat and lean clays. The levee is underlain by a thick (7-15ft) fat and lean clay blanket layer which is underlain by semi-pervious silt layer. The embankment, blanket and semi-pervious silt layer are underlain by a poorly graded sand and silty sand pervious aquifer.

### 5.1.4 YOLO BYPASS EAST LEVEE

On the Yolo Bypass east levee (Sta. 0+00 to Sta. 197+65) there is moderate riparian habitat (vegetation) on the existing waterside bench the majority of which are medium to large trees. There is very little landside vegetation existing near the levee toe or on the levee slope. Encroachments include fences, utility poles near the landside toe along the levee alignment, commercial/industrial developments, the I-80 freeway overcrossing, and railroad tracks. The levee crest surface is an aggregate road base with access points along the alignment within the adjacent commercial/industrial facilities.

The levee embankment is predominantly comprised of fat and lean clays. The levee is underlain by a fat and lean clay blanket layer varying in thickness (5-20ft) with discontinuous thin layers of poorly graded silty sands within the upper foundation which is underlain by semi-pervious silt layer. The embankment, blanket and semi-pervious silt layer are underlain by a poorly graded sand and silty sand pervious aquifer.

### 5.1.5 SACRAMENTO BYPASS NORTH LEVEE

The Sacramento Bypass north levee contains moderate riparian habitat (vegetation) adjacent to the levee embankment. The landside vegetation is very sparse, with little to no vegetation at the landside toe, or on the landside slope. On the waterside, there are notable amounts of large trees near the waterside berm and continuing out laterally into the channel for the majority of the alignment. Few encroachments are present along the alignment; nearing the upstream limit, a small pump station is adjacent to the landside levee slope. The levee crest surface is an aggregate road base with access gates at each end, east and west, of the alignment on County Rd. 126 .

The levee embankment is predominantly comprised of fat and lean clays throughout the alignment. The levee is underlain by a thick (15-20ft) lean and fat clay blanket layer which is underlain by a semi-pervious clayey sand of varied thickness. At the landside of the embankment, a berm was constructed of pit-run fill of predominantly cobbles and fine gravels with clay to aide in embankment stability.

The description of Sacramento Bypass North Levee is included to aid in explanation of the overall project area. Although the Sacramento Bypass North Levee, is not part of the federally authorized project nor a project levee, the overall project alternatives address a potential widening of the bypass and thus a discussion of the existing geotechnical properties is warranted.

## 5.2 WEST SACRAMENTO SOUTH BASIN

The South Basin of the West Sacramento Project includes levees on the south bank (left) of the Port of West Sacramento, west bank (right) of the Sacramento River from the Stone Lock structure and continues downstream to the South Cross Levee to the Yolo Bypass East Levee (left). The Deep Water Ship Channel west levee (right) is also included in the south basin which is located adjacent to the Yolo Bypass East Levee Table X-X displays data on the levee alignment for each channel.

**Table 5-2: West Sacramento - South Basin - Levee Properties**

| Channel | Begin Sta. | End Sta. | Crest Width (ft) | LS Leve Slope | WS Levee Slope | Levee Heigh t (ft) |
|---|---|---|---|---|---|---|
| PSL | 0+00 | 189+65 | 25-35 | 4.0-5.5:1 | 3.0-3.5:1 | 8-12 |
| SRWL | 0+00 | 332+70 | 25-35 | 1.75-2.25:1 | 2.0:1 | 15-25 |
| SCL | 0+00 | 65+00 | 15-20 | 3.0:1 | 2.75:1 | 15-20 |
| DWSCWL | 0+00 | 1133+14 | 20-30 | 4.0-6.0:1 | 4.0-6.0:1 | 20-30 |
| YBEL | 0+00 | 145+00 | 15-25 | 2.25-3.0:1 | 3.0-10.0:1 | 15-20 |

### 5.2.1 PORT SOUTH LEVEE

The Port South levee (Sta. 0+00 to Sta. 189+65) contains sparse riparian habitat (vegetation) adjacent to the levee embankment. There is very little landside vegetation existing near the levee toe or on the levee slope. On the waterside bench moderate vegetation exists, mostly trees lining the adjacent downstream portion near the Stone Lock structure. Encroachments include the Daniel C. Palmadessi bridge overcrossing, and commercial/industrial facility structures near the landside of the levee embankment. The levee crest surface is an aggregate road base with access points along the alignment within the adjacent developments including at the Barge Canal Access at the upstream limit of the alignment (Sta. 170+00).

The levee embankment is predominantly comprised of fat and lean clays. The levee is underlain by a thick (15-20ft) fat and lean clay blanket layer. The embankment and blanket layers are underlain by a poorly graded sand and silty sand pervious aquifer.

### 5.2.2 SACRAMENTO RIVER WEST LEVEE

On the Sacramento River west levee (Sta. 0+00 to Sta. 332+70) there is significant vegetation on the waterside bench which varies in thickness. Typically within the reach, the waterside bench becomes wider moving downstream at the Bee's Lake area, and then decreases sharply in width as the embankment is directly adjacent to the channel. In some areas, there is significant landside vegetation (mostly large trees) existing near the levee toe, or on the levee slope. On the landside, numerous encroachments including residential subdivisions, fence lines, driveways, and irrigation ditches exist throughout the alignment. The levee crest surface contains the roadway surface of the South River Road which is asphaltic concrete pavement with numerous access points across the alignment mostly at roadway intersections. The intersections include Lake Washington Blvd., Linden Rd., and Gregory Ave.

The levee embankment is predominantly comprised of poorly graded silty sands, silty sands, and silts. The levee is underlain by a silt and clay blanket layer (8-15ft) which is underlain by pervious poorly graded sand and silty sand aquifer.

### 5.2.3 SOUTH CROSS LEVEE

The South Cross levee (Sta. 0+00 to Sta. 65+00) contains moderate riparian habitat (vegetation) adjacent to the levee embankment. There is very little landside vegetation existing near the levee toe or on the levee slope. At the waterside bench, moderate vegetation exists, sporadic trees line the edge of the channel. Encroachments include residential homes, fencelines, and various outstructures near the landside of the levee embankment. The levee crest surface is an aggregate road base with access points at both the upstream and downstream limits of the alignments as well as various access ramps throughout the adjacent properties.

The levee embankment is predominantly comprised of fat and lean clays. The levee is underlain by a thick (15-20ft) lean and fat clay and silt blanket layer. The embankment and blanket layers are underlain by a poorly graded sand and poorly graded silty sand pervious aquifer.

### 5.2.4 DEEP WATER SHIP CHANNEL WEST LEVEE

The Deep Water Ship Channel west levee (Sta. 0+00 to Sta. 1133+14) contains sparse riparian habitat (vegetation) adjacent to the levee embankment on both the landside and waterside. There is a significant waterside bench throughout the alignment as the channel is offset from the levee centerline approximately 500ft. There are few encroachments throughout the alignment which include fences and utility poles. The levee crest surface is an aggregate road base with access points most prevalent near the upstream limit of the alignment.

The levee embankment is predominantly comprised fat and lean clays. The levee is underlain by a lean and fat clay blanket layer which varies in thickness (5-25ft). The embankment and blanket layers are underlain by a poorly graded sand and poorly graded silty sand pervious aquifer.

### 5.2.5 YOLO BYPASS EAST LEVEE

On the Yolo Bypass east levee (Sta. 0+00 to Sta. 145+00) there is very limited riparian habitat (vegetation) on the existing waterside bench the majority of which are medium to large trees. Encroachments include fences, utility poles near the landside toe along the levee alignment, residential developments, a pump station facility near the downstream limit of the alignment, and an irrigation ditch at the landside levee toe. The levee crest surface is an aggregate road base with access points along the alignment at the Jefferson Blvd. intersection, along with additional location adjacent to Marshall Rd. and the various commercial facilities near the levee embankment.

The levee embankment is predominantly comprised of fat and lean clays. The levee is underlain by a fat and lean clay blanket layer varying in thickness (10-20ft) which is underlain by semi-pervious silt layer. The embankment, blanket and semi-pervious silt layer are underlain by a poorly graded sand and silty sand pervious aquifer.

## 6.0 POTENTIAL FAILURE MODES

For the purposes of problem identification and alternatives analysis, several different failure modes have been evaluated for the without project condition. The failure modes included seepage (under and through), slope stability, erosion, overtopping and seismic.

## 6.1 SEEPAGE

Seepage is subdivided into two categories, seepage through the levee embankment (through-seepage) and seepage beneath the levee embankment through foundation layers (under-seepage). Through-seepage occurs when water from the river passes through a pervious levee and weakens the interior of the existing levee causing internal erosion and leads to slope instability or movement of embankment material. Concentrated under-seepage that carries silt and sand up to the surface through a more or less open channel in the top stratum (usually of clays and/or silts) is known as a sand boil. Active erosion of sand or other soils from under a levee or top stratum as a result of substratum pressure and concentration of seepage in localize d channels is known as piping. If the hydrostatic pressure in the pervious substratum landward of a levee becomes greater than the submerged weight of the top stratum, the excess pressure will cause heaving of the top stratum, or a rupture at one or more weak spots. This results in a concentration of seepage flow that may cause sand boils and/or underground piping as shown in Figure 6-1.

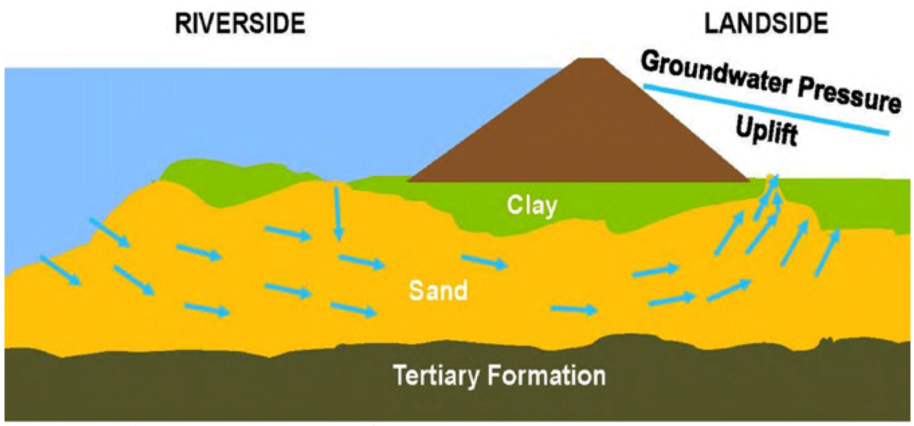



**Figure 6-1: Underseepage Distress**

## 6.2 SLOPE STABILITY

Hydraulic loading of the levee during a flood event reduces the strength of the levee embankment materials causing instability in the embankment slope. Additionally, uplift pressures caused by an excess in pore water pressure at the landside levee toe, can lead to the movement of embankment material within the levee due to seepage cause levee instability, as shown in Figure 6-2.

Levee instability can occur on both the waterside and landside of the embankment. Slope stability of the landside slope is typically analyzed and in instances where the waterside slope is somewhat steep, waterside slope stability may be analyzed as well. Cases will also exist where a levee is constructed of less permeable materials and rapid drawdown condition occurs. Rapid drawdown conditions arise when a submerged slope experiences a sudden reduction in water level. This change in water surface elevation causes a change in pore water pressure within the embankment having a low permeable material. The excess pore water pressure contained in the embankment may lead to a waterside slope stability failure. While waterside and rapid drawdown slope stability are potential failure modes, they typically have limited affect on feasibility level designs and are therefore considered design level analysis. Rapid drawdown slope failures pose different life safety risks as compared to landside slope failures and seldom dictate design. Stability failures can also occur due to erosion along the waterside bank progressing towards the levee embankment.

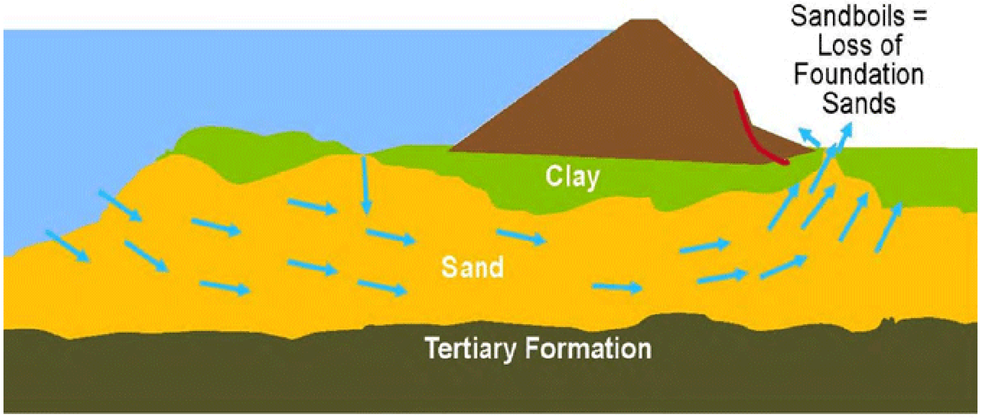



**Figure 6-2: Underseepage Induced Slope Instability Distress**

## 6.3 EROSION

Erosion is the wearing away of the riverbank and or waterside levee slope due to high flows. Erosion can also cause the degradation of the channel invert (scour) causing slope instability. Erosion can occur on the landside of the levee to due overtopping. Erosion occurs when the velocity of the river generates an effective hydraulic shear stress greater than the critical shear stress of the soil over which it flows. As the critical shear stress of the soil is exceeded, soil particle movement begins. As the amount of time the flow is applied, erosion will occur and the rate at which vary. Loosely compacted cohesionless soils are most susceptible to erosion; whereas cohesive engineered fill is less susceptible. Erosion events can also lead to catastrophic waterside bank and levee embankment stability failure as the time of applied flow increases throughout a flood event.

## 6.4 SEISMIC

Levees can fail as result of a seismic load which may cause degradation due to liquefaction. Liquefaction can lead to detrimental consequences such as loss of freeboard due to embankment instability, transverse crack-induced piping, and loss of freeboard due to settlement. Evaluations are typically completed to determine the liquefaction resistance of soils, this is known as liquefaction triggering. Other seismically induced failures include lateral spreading which can cause vertical displacement of the levee leading to loss of freeboard and levee stability.

## 6.5 OVERTOPPING

Overtopping occurs when the water surface elevation is greater than the elevation of the levee crest. In this case, water will flow over the crest, onto the landside of the levee. As the levee is overtopped, the action of the water flowing down the levee slope and into the basin may cause backside erosion of the landside levee slope and levee toe. This backside erosion may lead to sloughing of the levee and/or breeches.

## 7.0 CRITERIA

The following paragraphs will present USACE standard levee design and construction criteria as established in both national (HQ) and local (District and Division) policy documents and a discussion on how the PDT has made assumptions in applying those criteria to the West Sacramento project.

## 7.1 SEEPAGE AND SLOPE STABILITY

Seepage and slope stability vertical exit gradient and factor of safety criteria respectively for the geotechnical analysis that forms the basis of the geotechnical improvement measures were established based on ETL 1110-2-569 *Design Guidance for Levee Underseepage*, EM 1110-2-1913 *Design and Construction of Levees*, SOP-003, and the *Urban Levee Design Criteria*. Steady state seepage analysis for the water at the design elevation considered a maximum allowable vertical exit gradient at the toe of the levee to be less than 0.5. In general, this provides a factor of safety against uplift failure of about 1.60 considering the impervious blanket saturated unit weight of 112 pounds per cubic foot (pcf). Steady state seepage analysis for the water at the top of levee elevation considered a maximum allowable vertical exit gradient at the toe of the levee to be less than 0.8. In general, this provides a factor of safety against uplift failure of about 1.00 considering the impervious blanket saturated unit weight of 112 pounds per cubic foot (pcf). The minimum required factor of safety for the same design water surface elevation for the landside steady state slope stability analysis is 1.40. The minimum required factor of safety for the top of levee water surface elevation for the landside steady state slope stability analysis is 1.20. For landside seepage berms a maximum gradient of 0.8 is required at the berm toe. During construction, post construction, rapid drawdown, and waterside partial pool analysis cases were considered to be design level and were therefore not performed for this feasibility study.

## 7.2 EROSION

The Sacramento and American Rivers have well established susceptibility to erosion distress which has lead to several near levee failures. In general, there is no set of criteria for determining need for erosion improvements. However; the Sacramento River Bank Protection Program (SRBPP) since 1974 has prioritized critical erosion site repair. While the original method of site selection was simple field inspection, subsequent methodologies have adopted more quantitative selection criteria that have evolved over time. In 2007, Ayres Associates developed a Site Priority Ranking Report that account for several factors including; existing bank erosion in the levee prism, berm width less than 35 feet, bank slope, erosion length, as well as several other factors. In 2011, the Sacramento District updated the site priority ranking methodology.

## 7.3 SEISMIC

The main purpose of seismic vulnerability analyses was to identify the potential seismic performance of a levee. Although seismic remediation generally will not be implemented based on these analysis results, a levee's seismic degradation potential should be considered during selection of a static remediation, or in developing an emergency action plan to be implemented following an earthquake. Following an earthquake, a repair must be implemented to establish a 10yr level of protection within 8 weeks after the event.

Many levees are constructed over alluvial deposits, which may be susceptible to liquefaction or degradation by earthquakes. Levees meeting static stability criteria likely have sufficient factors of safety to resist the additional loading from earthquakes unless the levee or foundation materials lose significant strength due to liquefaction. Since many levees are infrequently loaded and thus the embankment is likely to be unsaturated at the time of a large earthquake, the material in the levee often can be considered non-liquefiable due to lack of saturation. As a result, the integrity of most levees following a strong earthquake is controlled by the liquefaction potential of its foundation soil.

Major concerns during and after a seismic event are transverse cracks that may develop between liquefied levee reaches and non-liquefied levee reaches and at locations where liquefied levee reaches contain or abut appurtenant structures with rigid or deep foundations. Such zones should be identified and given special attention.
For the most critical category of levee (e.g., urban levees that are frequently hydraulically loaded) the following displacements are acceptable:

- Any deformation inducing crest displacement of 1 foot or less, unless larger lateral movements comprise the ability of foundation cut-offs or toe drains, etc. to provide for safe retention of high water.
- If more than 1 foot of seismic displacement is predicted, deformation is still acceptable if the levee continues to ensure water retention with 0.3 m or 3 feet of freeboard for a 200-year flood event.
- If other safety criteria are met (e.g., cracking that can be repaired in a few days).

## 7.4 GEOMETRY

The typical USACE levee section, established by EM 1110-2-1913, is nationally considered to have a minimum 10-foot crest with waterside and landside slopes not steeper than 2:1 (horizontal: vertical). According to the Sacramento District 1969 "Design Manual for Levee Construction" levees should be constructed with 3:1 waterside and 2:1 landside slopes with either a 20 or 12-foot levee crest width for main stream or tributary levees respectively. The use of Sacramento District standard sections is generally limited to levees of moderate height, less than 25 feet, in reaches where there are no serious underseepage problems, weak foundation soils, or constructed of unsuitable materials. The standard levee section may have more than the minimum allowable factor of safety relative to slope stability and seepage, its slopes being established primarily on the basis of construction and maintenance considerations. The SOP-003, suggests a 20-foot crest width with 3:1 waterside and landside slopes except existing levees with good past performance exists where existing 2:1 slopes are acceptable. The SOP-003 accepted a reduced crest width of 15 feet for levees along minor creeks or minor tributaries.

### 7.5 VEGETATION, ENCROACHMENT, AND ACCESS

Vegetation, encroachment, and access policy includes EM 1110-2-1913, SOP-003, and ETL 1110-2-571 *Guidelines for Landscaping and Vegetation Management at Levees, Floodwalls, Embankments Dams, and Appurtenant Structures*. The vegetation-free zone, as established by ETL 1110-2-571, is a three-dimensional corridor surrounding all levees, floodwalls, and critical appurtenant structures in a flood damage reduction system. The vegetation-free zone applies to all vegetation except grass. The minimum height of the corridor is 8 feet, measured vertically from any point on the ground. The minimum width of the corridor is the width of the flood-control structure (Levee toes or floodwall stem), plus 15 feet on each side, measured from the outer edge of the outermost critical structure. Figure 7-1 is a representation of the vegetation-free zone of a basic levee cross-section.

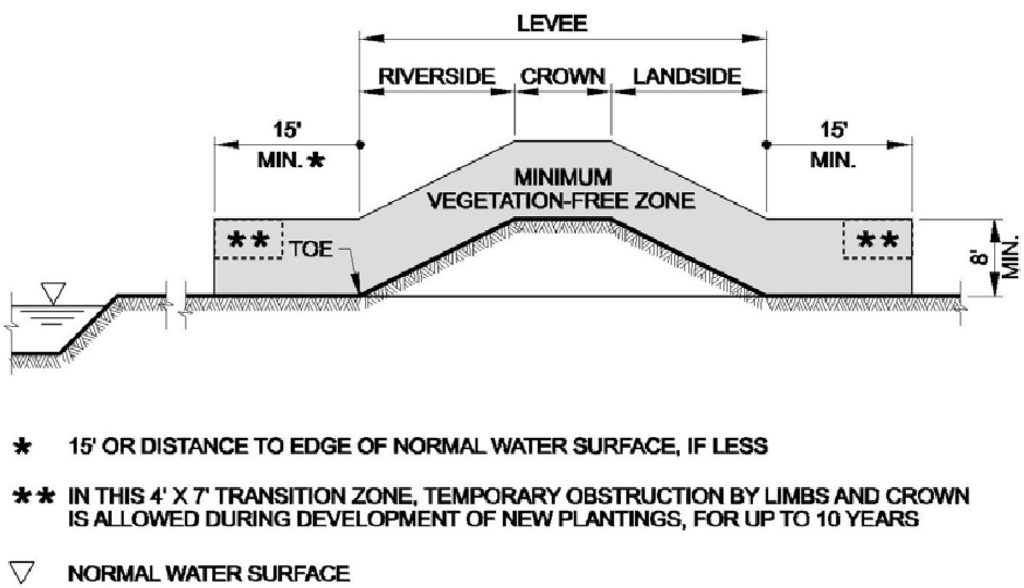



**Figure 7-1: Vegetation-Free Zone of Basic Levee**

The primary purpose of the vegetation-free zone is to prevent any damages of the levee embankment due to vegetation (including seepage along the woody vegetation root system, additional scouring of the waterside slope due to trees uprooting, and attraction of rodents) and to provide a reliable corridor of access to and along the flood-control structure for flood fighting, inspection and maintenance of the flood control structures. This corridor must be an all weather access and free of obstructions to assure adequate access by personnel and equipment for surveillance, inspection, maintenance, monitoring, and flood-fighting. In the case of flood-fighting, this access corridor must also provide the unobstructed space needed for the construction of temporary flood-control structures. Access is typically by four-wheel-drive vehicle, but for some purposes, such as maintenance and flood-fighting, access is required for larger equipment, such as tractors, bulldozers, dump trucks, and helicopters. Accessibility is essential to the reliability of flood damage reduction systems. SOP-003 established a minimum landside levee toe access width of 20 feet for newly constructed levees. The EM 1110-2-193 however does not specify the corridor width for access along the levee, it requires only access to be provided on the levee slopes and crest.

For a levee section to be considered compliant with USACE vegetation policy it must either have been cleared of vegetation within the vegetation free zone or eligible for a variance from USACE policy on vegetation in ETL 1110-2-571. Since the publication of ETL 1110-2-571, a Policy Guidance Letter (PGL) has been developed stating that waterside planting berm is acceptable. The variance must assure that safety, structural integrity, and functionality are retained, and accessibility for maintenance, inspection, monitoring, and flood-fighting are retained. The variance may require structural measures to mitigate vegetation, such as overbuilt sections, to improve levee system reliability, redundancy, or resiliency with respect to the detrimental impacts of the vegetation.

## 8.0 TYPICAL IMPROVEMENT MEASURES

Where levee height, geometry, erosion, access, vegetation, seepage, and slope stability deficiencies were identified (criteria not met) improvement measures consisting of cutoff walls, seepage berms, relief wells, stability berms, earth reinforcement, flattened embankment slopes, flood walls, retaining walls, sliver fills, riprap slope protection, and various other measures were included in development of conceptual alternative cross-sections. This section of the report discusses the various different improvement measures considered at a conceptual level, and not as applied to a specific reach.

### 8.1 UNDERSEEPAGE

#### 8.1.1 CUTOFF WALLS

Seepage cutoff walls are vertical walls of low hydraulic conductivity material constructed through the embankment and foundation to cut off potential through seepage and underseepage. In order to be effective for underseepage mitigation, cutoff walls usually tie into an impervious layer. Cutoff walls generally require no additional permanent levee footprint. The crown of the levee should be degraded by one third of the levee height or as much as necessary to provide sufficient working surface (minimum 35 feet) and prevent hydraulic fracture of the levee. The levee would then be rebuilt either with the existing levee material and an impervious cap above the cutoff wall or with imported impervious levee fill material. Cutoff walls are typically constructed of either a soil bentonite (SB), soil cement bentonite (SCB), or cement bentonite (CB) mixture depending on in-situ soil conditions and desired construction method.

The conventional slurry method is an open trench method that uses an excavator with a long-stick boom to excavate the slurry trench. A bentonite-water slurry is used to keep the trench open and stable prior to backfilling. Soil from excavation or borrow area is mixed with bentonite (or with cement and bentonite) then pushed into the trench, displacing the bentonite-water slurry. The cutoff wall trench can also be backfilled with self-hardening slurry mixture (cement-slag-bentonite). The self-hardening slurry backfill can be used to keep the trench open and stable allowing excavation of a new section without waiting for the entire trench to be excavated. The conventional method using a long stick and boom excavator has a maximum depth of 70 to 80 feet. Deeper cutoff walls, up to about 150 feet could be excavated using cable excavation method with crane rigs.

Mix-in-place methods of cutoff wall construction include deep mixing method, jet grouting, and cutter soil mixing. Deep Mixing Method uses specialized construction equipment to mix the soil with bentonite and cement in situ and is capable of depths more than 100 feet. Jet grouting uses the injection of high pressure grout to create soil-cement-bentonite mixtures in overlapping columns or panels within the subsurface soils. Cutter soil mixing uses a cutter head with typically two cutter wheels around a horizontal axis that allows vertical penetration within the subsurface soils. Bentonite and/or cement slurry are injected during the penetration and withdrawal of the cutter head. Like jet grouting, overlapping primary and secondary panels is necessary to complete the cutoff wall.

### 8.1.2 RELIEF WELLS

Pressure relief wells relieve excess pore pressures that can build up beneath a surficial impervious blanket layer to reduce exit gradient. Relief wells collect seepage and bring it to the surface where it can be discharged freely on the ground surface or collected and drained away from the levee toe. Drainage from relief wells can either be into an existing (sewers or roadways) or proposed drainage system necessitating either gravity flow or potentially requiring pumping facilities. Relief wells usually require long term maintenance to ensure they operated efficiently. In general, the maintenance required to retain efficiency, require capacity in existing urban interior drainage systems, and may not be suitable for all types of soil stratigraphy. The operations and maintenance program increases the long term costs, however the application of relief wells in certain cases may still be cost effective as compared to alternative improvement measures.

### 8.1.3 SEEPAGE BERMS

Seepage berms are earth structures built at the landside toe that provide additional weight to prevent blanket layer heave, reduce exit gradients, and can allow safe exit of underseepage. The minimum seepage berm width is typically four times the levee height and the maximum width is generally 300 to 400 feet. Minimum thickness at the levee toe is typically 5 feet and 3 ft at the berm toe. Seepage berms can be pervious, semi-pervious, or impervious and require a significant amount of land. For urban areas, due to adjacent property uses, there is not sufficient room on the landside toe for a seepage berm without real estate impacts and without relocations.

## 8.2 SLOPE STABILITY

### 8.2.1 SLOPE FLATTENING

Slope flattening is a structural method to reinforce unstable slopes. Both the waterside and landside slopes can be re-graded using construction equipment. In most cases, this process requires the removal of all vegetation and encroachments from the levee slope being flattened. Slopes are typically flattened to 3H:1V to 5H:1V.

### 8.2.2 STABILITY BERMS

Stability berms are constructed of a random fill material placed on the levee slope to increase the slope stability. These berms may be constructed of any compacted random material placed on a chimney drain along the existing levee slope connected to a drainage blanket underneath the berm to capture the seepage through the levee and drain it outside the levee prism, or, if seepage through the levee is not an issue, it can be constructed directly over the levee slope as needed to increase the slope stability only. In case a chimney drain is used a thin filter sand layer is placed between the drainage layer and the levee embankment and native soils. Geotextile fabric may be placed between the free drainage layer and the levee fill. Typically the height of the stability berm in 2/3$^{rd}$ of the height of the levee or to the design water surface elevation (WSE) and extends for approximately 15 ft in width or as determined by the structural needs of the levee along that reach.

## 8.3 HEIGHT

### 8.3.1 FLOODWALL/RETAINING WALL

Floodwalls are an efficient, space-conserving method for containing unusually high water surface elevations. They are often used in highly developed areas, where space is limited. They are primarily constructed from pre-fabricated materials, although they may be cast or constructed in place. Floodwalls consist of relatively short elements constructed on the levee crest, making the connections very important to their stability. Floodwalls are typically located along a levee waterside hinge point to allow vehicular access along the crown. The drawback is that floodwalls prohibit access to or from the slopes, and may inhibit visual inspection of the waterside slope and toe areas from the crown if the wall is of sufficient height during inspection.

### 8.3.2 EMBANKMENT FILL

To address deficiencies found in the required levee freeboard various methods of raising the existing levee crown elevation could be implemented. The two most likely alternatives include a crown-only raise and a full levee raise. A crown only levee raise assumes that the levee crown is currently wide enough to support the placement of additional embankment material while maintaining the minimum allowable crown width and slopes upon the completion of the raise. A full levee raise includes an embankment raise from the waterside crown hinge point upward at a 3H:1V slope, establishing a new crown width, and then down the landside at a new 3H:1V slope.

## 8.4 EROSION

### 8.4.1 LAUNCHABLE ROCK TRENCH

To protect against waterside erosion in areas where a waterside berm exists, a launchable rock trench may be constructed. The intent of the trench is to prevent further waterside erosion into the levee embankment particularly at the waterside levee toe. This is accomplished by placing rip-rap a certain height on the waterside slope and excavating a trench at the waterside toe, or where the waterside slope meets the berm. Rip-rap is then placed in the trench and then covered with random fill. As the waterside berm is eroded, it will eventually reach the launchable rock trench. At this point, the undermining action of the erosion event and soils surrounding the trench will allow for the rip-rap contained in the trench to “launch” into the void created adjacent to the trench. The rip-rap previously contained in the trench will protect against further erosion landward in to the levee embankment.

### 8.4.2 BANK PROTECTION (ON-BANK AND ON-SLOPE)

In areas that have no or minimal waterside berm, rip-rap is placed on the waterside levee slope to protect against erosion. This entails filling the eroded portion of the bank and installing stone protection along the levee slope from the base of the erosion area to the top of the erosion area. Vegetation would be limited to grass. If there is a natural bank distinct from the levee that requires erosion protection, it would be treated with stone protection. Existing vegetation would be removed within the vegetation free zone. Grass would be allowed in this area.

Additionally a rip-rap waterside berm could be constructed from the base of the erosion to above the mean summer water surface level (MSWL) and then placing stone protection on the levee or bank slope above the MSWL. The rock berm would support riparian vegetation and provide a place to anchor in-stream woody material (IWM). This design provides near-bank, shallow-water habitat for fish.

## 8.5 GEOMETRY, VEGETATION, ACCESS, AND ENCROACHMENTS

### 8.5.1 STANDARD LEVEE GEOMETRY

The levee needs to be regaraded to the minimum requirements of the SOP003. The minimum levee section for new construction should have a 3H:1V waterside slope, minimum crest width of 20 feet for mainline levees, major tributary levees, and bypass levees; a minimum of crest width of 12 feet for minor tributary levees, and a 3H:1V landside slope as required in SOP-003. Existing levees with landside slopes as steep as 2H:1V may be used in rehabilitation projects if the landside slope performance has been good and if the slope stability analyses determined the factors of safety are adequate.

### 8.5.2 TOE ACCESS

The purpose of the toe access easement is to allow for necessary maintenance, inspection, and floodfight access. SPK guidance in SOP 003 requires a 20 ft. wide easement landside of the levee for new levees as well for existing levees. Research throughout the USACE districts concluded that the minimum toe access required in most applications was 10 ft. This 10 ft. width would accommodate an all weather road along the landside levee toe.

### 8.5.3 VEGETATION

The design effort will completed to comply with the USACE vegetation policy. Where vegetation management standards do not meet the ETL requirements, a variance may be approved to a levee system or portion of that system to provide for the same levee functionality as intended in ETL 1110-2-571. In consideration for a vegetation variance request (VVR), the VVR will preserve, protect, and enhance the natural resources of the levee system or segment. The requester must demonstrate that a variance is the only reasonable means to achieve the required criteria as stated in ETL 1110-2-571. A more detailed description of the requirements and process for requesting the vegetation variance can be found in the above stated ETL and associated policy guidance letters (PGL).

### 8.5.4 PLANTING BERMS

Planting berms can be both on the waterside and landside of the levee. The difference is that landside planting berms are allowed by the ETL and waterside planting berm have to be approved as a variance from the ETL. These berms are additional cross sectional areas required to accommodate desired vegetation. It preserves access and protects the prism from root-related damage.

### 8.5.5 ENCROACHMENTS

Encroachments are reviewed on a case-by-case basis. Encroachment types may vary from fences, non-permitted access gates, staircases, gardens, irrigation systems, lighting and various other occurrences adjacent to, at the levee toe, or on the landside/waterside levee slope. If an encroachment inhibits inspection or maintenance activities of the levee, consideration should be given to removing or relocating the encroachment to allow proper maintenance and inspection.

## 8.6 SACRAMENTO WEIR AND BYPASS WIDENING

The existing Sacramento Weir and Bypass, which allow high flows in the Sacramento River to be diverted into the Yolo Bypass, could be expanded to accommodate increased bypass flows. The increased flows from the Sacramento River to the Yolo Bypass would serve to reduce the stage on the levees downstream thereby negating a potential need for levee raises. The existing north levee of the Sacramento Bypass would be degraded and a new levee constructed to the north. The existing Sacramento Weir would be expanded to match the wider bypass.

## 8.7 DEEP WATER SHIP CHANNEL CLOSURE STRUCTURE

Construction of an operable closure structure on the Deep Water Ship Channel located just downstream of the Port South levee and Yolo Bypass East Levee (South Basin) cconfluence is being examined. The structure would include multiple gates to be operated allowing both flows in and out of the north basin providing a level of protection comparable to other improvement measures. The cross channel structure would also incorporate tie-in levees to the existing embankments of the Yolo Bypass East Levee and the Deep Water Ship Channel West Levee with the use of T-walls and/or levees. A closure structure of this nature is similar to an evaluation completed by USACE 2012 would evaluated the feasibility of constructing a closure structure near the I Street Bridge on the Sacramento River. Similar considerations with respect to cost and constructability should be taken in this application as well.

## 9.0 CROSS-SECTION SELECTION

Cross-sections for geotechnical analysis were selected to represent critical surface and subsurface conditions of each reach. The topography of each reach is inherently variable. The existence of access ramps on both landside and waterside of the levee, railroads running perpendicular and parallel to the levee, and/or pump stations or other structures built up adjacent to the levee section create difficulties to discern the typical versus critical cross-section. The sections were selected based on subsurface data, laboratory test results, geomorphology, surface conditions, field reconnaissance, historical performance, and levee geometry. The ground surface elevations used in the cross-sections were based on a LiDAR and topographical survey completed in November 2008 for the DWR, ULE project. The natural soil layers were delineated based on boring logs and laboratory test results. Cross-sections of existing levee geometry and subsurface conditions at each index point are included as Enclosure 3.

Typically one cross section per reach was selected for analysis and is referred to as an index point. Within each reach the same index point is used in hydraulic, economic, and geotechnical analysis. In some cases, multiple cross sections were analyzed in each reach to verify the initial location. Table 9-1 presents the cross-sections where geotechnical analyses were performed, not all were incorporated into the economic analyses which would be referred to as index points.

**Table 9-1: Geotechnical Analysis Locations**

| Basin | Location | Bank | River Mile | Sta. | Economic Analyses |
|---|---|---|---|---|---|
| NORTH | Port North Levee | North | 42.83 | 117+37 | N |
| NORTH | Sacramento Bypass South Levee | South | 1.6 | 32+00 | N |
| NORTH | Sacramento Bypass South Levee | South | 1.6 | 52+00 | Y |
| NORTH | Sacramento River West Levee | West | 61.67 | 96+00 | Y |
| NORTH | Sacramento River West Levee | West | 60.20 | 190+00 | Y |
| NORTH | Yolo Bypass East Levee | East | 41.90 | 36+00 | N |
| NORTH | Yolo Bypass East Levee | East | 43.10 | 107+31 | Y |
| NORTH | Sacramento Bypass North Levee | North | 0.4 | 8+30 | N |
| SOUTH | Deep Water Ship Channel West Levee | West | 41.21 | 12+00 | Y |
| SOUTH | Port South Levee | South | 43.45 | 123+55 | Y |
| SOUTH | South Cross Levee | South | 38.25 | 17+50 | N |
| SOUTH | Sacramento River West Levee | West | 56.74 | 264+00 | Y |
| SOUTH | Sacramento River West Levee | West | 53.08 | 80+00 | Y |
| SOUTH | Sacramento River West Levee | West | 51.07 | 35+22 | N |
| SOUTH | Yolo Bypass East Levee | East | 40.82 | 10+00 | N |
| SOUTH | Yolo Bypass East Levee | East | 37.22 | 53.96 | N |

## 10.0 HYDRAULIC LOADING CONDITIONS

Water surface profiles for the West Sacramento study area were obtained from the Hydraulics and Hydrology Branch, Sacramento District. The profiles provide water surface elevations in NAVD 88 by river mile for various flood frequencies. Deterministic seepage and stability analyses were performed for various flood frequencies typically incorporating the 10yr, 25yr, 50yr, 100yr, 200yr, 500yr, and top of levee. The probabilistic analyses were performed for a range of stages not correlated to flood frequency, but which represented stages from no head (landside toe of levee) to maximum head (top of levee). Tables 10-1 and 10-2 below summarize the water surface elevations deterministically analyzed at each index point, by basin.

**Table 10-1: West Sacramento - North Basin Analyses Water Surface Elevations**

| Index Point | Event | Stage | Head | Index Point | Event | Stage | Head |
|---|---|---|---|---|---|---|---|
| PNL_STA_117+37 | Crest | 22.2 | 5.19 | SRWL_STA_96+00 | Crest | 40.90 | 18.50 |
| | 500yr | 22.28 | N/A | | 500yr | 38.19 | 15.78 |
| | 200yr | 20.93 | 3.90 | | 200yr | 36.17 | 13.76 |
| | 100yr | 19.83 | 2.80 | | 100yr | 34.71 | 12.30 |
| | 50yr | 18.71 | 1.68 | | 50yr | 34.03 | 11.62 |
| | 25yr | 17.78 | 0.65 | | 25yr | 33.49 | 11.08 |
| SBSL_STA_ 32+00 | Crest | 36.63 | 20.85 | SRWL_STA_190+00 | Crest | 39.47 | 11.47 |
| | 500yr | 35.95 | 20.17 | | 500yr | 38.27 | 10.27 |
| | 200yr | 34.38 | 18.60 | | 200yr | 36.14 | 8.14 |
| | 100yr | 33.04 | 17.26 | | 100yr | 34.66 | 6.66 |
| | 50yr | 32.23 | 16.45 | | 50yr | 33.95 | 5.95 |
| | 25yr | 31.42 | 15.64 | | 25yr | 33.36 | 5.36 |
| YBEL_STA._36+00 | Crest | 37.15 | 17.79 | SBNL_STA_8+30 | Crest | 36.00 | 19.16 |
| | 500yr | 33.20 | 13.84 | | 500yr | 34.53 | 17.69 |
| | 200yr | 32.25 | 12.89 | | 200yr | 33.36 | 16.52 |
| | 100yr | 31.22 | 11.86 | | 100yr | 32.16 | 15.32 |
| | 50yr | 30.32 | 10.96 | | 50yr | 31.24 | 14.40 |
| | 25yr | 29.41 | 10.05 | | 25yr | 30.33 | 13.49 |

**Table 10-2: West Sacramento - South Basin Analyses Water Surface Elevations**

| Index Point | Event | Stage | Head |
|---|---|---|---|
| DWSCWL_STA_12+00 | Crest | 34.44 | 31.94 |
| | 500yr | 22.28 | 19.78 |
| | 200yr | 20.92 | 18.42 |
| | 100yr | 19.83 | 17.33 |
| | 50yr | 18.71 | 16.21 |
| | 25yr | 17.68 | 15.18 |
| SCL_STA_17+50 | Crest | 27.55 | 18.74 |
| | 500yr | 33.98 | N/A |
| | 200yr | 32.29 | N/A |
| | 100yr | 30.89 | N/A |
| | 50yr | 30.32 | N/A |
| | 25yr | 29.65 | N/A |
| | 10yr | 27.01 | 18.2 |
| SRWL_STA_264+00 | Crest | 40.52 | 20.90 |
| | 500yr | 36.50 | 16.88 |
| | 200yr | 34.53 | 14.91 |
| | 100yr | 33.08 | 13.46 |
| | 50yr | 32.41 | 12.79 |
| | 25yr | 31.83 | 12.21 |
| SRWL_STA_80+00 | Crest | 39.00 | 21.44 |
| | 500yr | 34.71 | 17.15 |
| | 200yr | 32.93 | 15.37 |
| | 100yr | 31.53 | 13.97 |
| | 50yr | 30.86 | 13.30 |
| | 25yr | 30.28 | 12.72 |

| Index Point | Event | Stage | Head |
|---|---|---|---|
| PSL_STA_123+55 | Crest | 21.67 | 14.50 |
| | 200yr | 20.93 | 13.76 |
| | 100yr | 19.83 | 12.66 |
| | 50yr | 18.71 | 11.54 |
| | 25yr | 17.68 | 10.51 |
| SRWL_STA_35+22 | Crest | 34.65 | 19.92 |
| | 500yr | 33.48 | 18.75 |
| | 200yr | 31.85 | 17.12 |
| | 100yr | 30.47 | 15.74 |
| | 50yr | 29.81 | 15.08 |
| | 25yr | 29.23 | 14.50 |
| YBEL_STA_10+00 | Crest | 31.93 | 22.04 |
| | 500yr | 32.86 | N/A |
| | 200yr | 31.93 | 22.04 |
| | 100yr | 30.92 | 21.03 |
| | 50yr | 30.03 | 20.14 |
| | 25yr | 29.13 | 19.24 |
| YBEL_STA_53+96 | Crest | 32.71 | 32.28 |
| | 500yr | 31.15 | 30.72 |
| | 200yr | 30.26 | 29.83 |
| | 100yr | 29.29 | 28.86 |
| | 50yr | 28.42 | 27.99 |
| | 25yr | 27.53 | 27.10 |

## 11.0 SEEPAGE AND SLOPE STABILITY ANALYSIS

### 11.1 STEADY STATE SEEPAGE ANALYSIS METHODOLOGY

Deterministic steady state seepage analysis was performed using SEEP2D within GMS 6.5 (Groundwater Modeling System), a finite element program. Results from the seepage analysis were used to calculate average vertical exit gradients at the landside levee toe and/or at a more critical location near the levee toe if applicable, for example at the invert of the empty drainage ditch. The pore pressures and/or phreatic surfaces were exported to UTEXAS4.0 for use in slope stability analysis.

Boundary conditions along the waterside ground surface from the waterside model extents to the levee slope were assigned as fixed total head conditions corresponding to the analyzed water elevation. On the landside, exit face boundary conditions are applied from the crest hinge point to landside extents of the model. All other boundaries not explicitly assigned a condition are assumed by the program to be no flow which include both vertical faces of the model and the bottom nodes. The landside model extents were extended 2,000 feet from the levee centerline and to the end of available topographic information on the waterside which includes bathymetric information when available. Figure 11-1 shows a typical GMS SEEP2D seepage model.

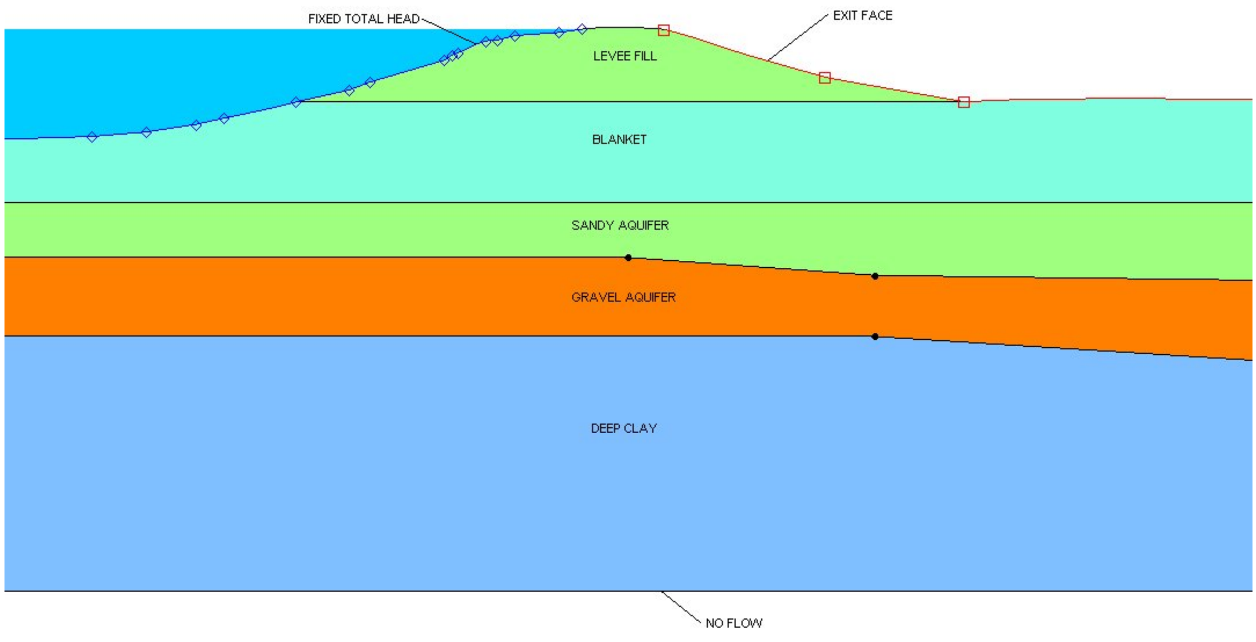



**Figure 11-1: Typical GMS SEEP2D Seepage Analysis Model**

Levees constructed either of fine grained clays, having stability berms with drainage layers extended along the levee slope that captures any seepage through the levee, or having cutoff walls constructed through the levee embankment are unlikely to be susceptible to through-seepage caused internal erosion. Levees of silt, silty sand, and sand were considered to be susceptible to internal erosion caused by through seepage and could potentially be considered as deficient from a through seepage perspective.

## 11.2 STEADY STATE SLOPE STABILITY ANALYSIS METHODOLOGY

Embankment slope stability against shear failure was analyzed using the UTEXAS4.0 software package for steady state conditions. Analyses to find factors of safety against sliding were conducted using a floating grid automatic circular failure surface search routine to identify the critical failure surfaces with Spencer Procedure within the embankment and/or foundation. The Spencer Procedure satisfies both force and moment equilibrium for each slice. A minimum weight restriction was applied to the slices within the failure surface to eliminate surficial failure surfaces. Where tensile stresses are expected on the failure surface due to the nature of the material (clay usually is producing cracks during dry weather), a crack with water to a certain depth in the crack was considered to eliminate the tensile stresses, but not compressive stresses. The appropriate depth for a crack is the one producing the minimum factor of safety, which corresponds to the depth where tensile, but no compressive, stresses are eliminated. If a crack was required, the maximum crack depth was set to producing the lowest factor of safety, typically two to four feet. Figure 11-2 shows a typical UTEXAS4.0 model.

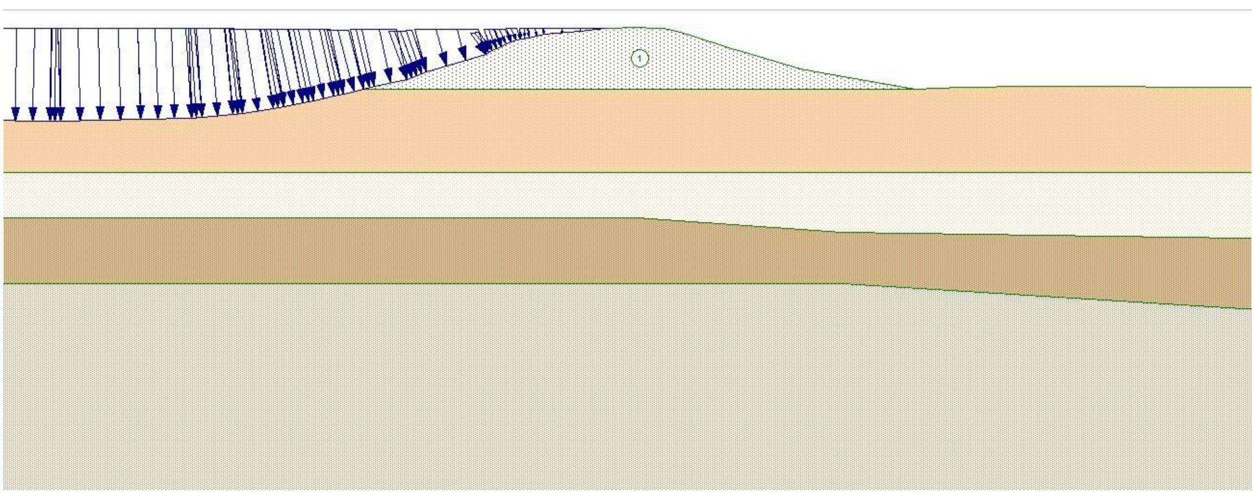


**Figure 11-2: Typical UTEXAS4.0 Slope Stability Analysis Model**

The long term evaluation with steady state seepage based on the assumption of a fully developed phreatic surface through the embankment was considered. Saturated unit weights are used in the embankment and the pore water pressure is imported from SEEP2D. External water pressures from the channel are applied as a distributed load against the landside slope. Effective shear strength parameters c' and Φ' were used for all materials.

## 11.3 MATERIAL PROPERTIES

Material properties including hydraulic conductivity for seepage analysis and drained (effective) shear strength and unit weight for slope stability analysis were determined based on field and laboratory data that was then generalized into appropriate parameters by material type. The stratigraphy of the existing levee cross-section was divided into unique layers typically consisting of levee embankment fill, foundation or blanket layer, pervious aquifer layers separated by an aquitard, and a deeper fine grained layer. Analysis material parameters were assigned considering saturated conditions.

From the generalized parameters, conservative seepage and slope stability analysis parameters were developed for the soil layers based on regression of site-specific field and laboratory test results and correlations at the location of the analyzed cross-section. Specific correlations included SPT blow counts, CPT tip resistance and sleeve frictions, Atterberg Limits, consolidations testing, and grain size distribution tests. Less conservative values (higher strength and lower hydraulic conductivity) were often present in individual tests or soil layers/borings; however, uncertainty exists in the field and laboratory testing based on the spacing between explorations, frequency of testing, appropriateness of correlations, and limitations of field and laboratory testing methods. The hydraulic conductivities, shear strengths, and unit weights used in the seepage and slope stability analysis are included as Enclosure 2.

Hydraulic conductivities were assigned based on soil classification and fines content using typical values developed and evolved from soil index property and hydraulic conductivity testing on samples gathered by the many subsurface investigations coupled with limited in-situ testing and engineering judgment performed by USACE, DWR, URS, Kleinfelder, and others on similar levees and in similar geologic conditions to this project. These values have been adapted for this project and are presented in Table 11-1 below. Prior to being used in analysis, the hydraulic conductivities presented in Table 11-1 were compared to sieve analysis and hydrometer correlations such as Kozeny-Carmen (Chapius, 2003), Chapuis's empirical equation (Chapuis, 2004), Hazen (extended by Chapuis, 2004), and the United States Bureau of Reclamation (USBR, 2011).

Most soil deposits have a different horizontal hydraulic conductivity than vertical hydraulic conductivity. The ratio of horizontal hydraulic conductivity divided by vertical hydraulic conductivity is referred to as anisotropy ratio ($K_H/K_V$). Anisotropy between horizontal and vertical conductivities is influenced by a number of factors including a variation in material properties within a modeled layer (interbedded lenses of sand in a silt or clay layer), cracks within the layer, etc. The analyses were performed using a soil anisotropy ratio of 4 for most naturally deposited layers. Thin clay blankets were given an anisotropy of 1 to 0.10 (assumed to be cracked) and some sands and gravels were given an anisotropy of 10.

**Table 11-1: Hydraulic Conductivities**

| Material Type | Soil Description | Hydraulic Conductivity | | | | |
|---|---|---|---|---|---|---|
| | | $K_H$ (cm/sec) | $K_H$ (ft/day) | $K_H/K_V$ | $K_V$ (cm/sec) | $K_V$ (ft/day) |
| Cutoff Wall | SCB, SB, CB | 1.0E-06 | 0.0028 | 1 | 1.0E-06 | 0.0028 |
| Clay | Engineered Embankment | 1.0E-06 | 0.0284 | 1 | 1.0E-0.6 | 0.0284 |
| | Non-Engineered Embankment | 1.0E-05 | 0.0284 | 4 | 2.5E-06 | 0.007 |
| | Blanket ≥10ft Thick or Embankments | 1.0E-05 | 0.0284 | 4 | 2.5E-06 | 0.007 |
| | Blanket 5ft<>10ft Thick | 1.0E-05 | 0.0284 | 1 | 1.0E-05 | 0.0284 |
| | Blanket ≤5ft Thick | 1.0E-05 | 0.0284 | 0.10 | 1.0E-04 | 0.284 |
| Silt | Elastic (plastic) | 5.0E-05 | 0.14 | 4 | 1.3E-05 | 0.035 |
| | Non-plastic | 2.0E-04 | 0.57 | 4 | 5.0E-05 | 0.14 |
| Clayey Sand to Sand | 30-49% fines | 5.0E-05 | 0.14 | 4 | 1.3E-05 | 0.035 |
| | 13-29% fines | 1.0E-04 | 0.28 | 4 | 2.5E-05 | 0.071 |
| | 8-12% fines | 1.0E-03 | 2.8 | 4 | 2.5E-04 | 0.71 |
| | 0-7% fines | 5.0E-03 | 14 | 4 | 1.3E-04 | 3.5 |
| Silty Sand to Sand | 30-49% fines | 5.0E-04 | 1.4 | 4 | 1.3E-04 | 0.35 |
| | 13-29% fines | 1.0E-03 | 2.8 | 4 | 2.5E-04 | 0.71 |
| | 8-12% fines | 5.0E-03 | 14 | 4 | 1.3E-03 | 3.5 |
| | 0-7% fines | 1.0E-02 | 28 | 4 | 2.5E-03 | 7.1 |
| Gravel | 28-49% fines | 4.0E-04 | 1.13 | 4 | 1.0E-04 | 0.28 |
| | 18-27% fines | 1.0E-03 | 2.8 | 4 | 2.5E-04 | 0.71 |
| | 13-17% fines | 6.0E-03 | 17 | 10 | 6.0E-04 | 1.7 |
| | 8-12% fines | 1.2E-02 | 34 | 10 | 1.2E-03 | 3.4 |
| | 0-7% fines | 2.5E-02 | 71 | 10 | 2.5E-3 | 7.1 |
| Gravel with Cobbles and Sand | 28-49%fines | 4.0E-04 | 1.13 | 4 | 1.0E-04 | 0.28 |
| | 18-27% fines | 1.0E-03 | 2.8 | 4 | 2.5E-04 | 0.71 |
| | 13-17% fines | 1.0E-02 | 28 | 10 | 1.0E-03 | 2.8 |
| | 8-12% fines | 1.0E-01 | 284 | 10 | 1.0E-02 | 28 |
| | 0-7% fines | 2.0E-01 | 570 | 10 | 2.0E-02 | 57 |
| Drain Rock | Gravel | 1.0E01 | 2835 | 1 | 1.0E01 | 2835 |

The resistance to penetration of the soils measured in blows per foot (field N-value) during the driving of Standard Penetration Test (SPT) samplers and Cone Penetrometer Test (CPT) tip resistance served as a site specific data source for the determination of shear strength parameters for granular, cohesionless soils through empirical correlations. Empirical correlations with SPT N-values by Uchida (1996) and Peck (1974) were used for the estimation of the drained (effective stress) angle of internal friction $\Phi'$. For cohesive soils (including clays and plastic silts), the empirical correlations by Mitchell (1976) and Bowles (1996) were used for estimation of $\Phi'$ using the Plasticity Index (PI) of the soil. Correlation values were compared with available shear strength laboratory testing.

For both cohesive and cohesionless materials, the shear strengths selected for analysis were typically equal to or less than the 1/3$^{rd}$ percentile of the data set. Shear strengths predicted by correlations were compared to typical published values and values used in previous analysis in similar materials, and then adjusted based on engineering judgment. Typical shear strengths, by material classification, used in steady state slope stability analysis are shown in Table 11-2.

**Table 11-2: Shear Strength of Soils**

| **Material Type** | **Soil Description** | **Shear Strength** | | |
|---|---|---|---|---|
| | | **$C'$ (psf)** | **$\Phi'$ ($^o$)** | **$\gamma$(pcf)** |
| Cutoff Wall | SB | 50 | 0 | 85 |
| | SCB | 500 | | |
| | CB | 5000 | | |
| Clay | Clay Foundation | 50-100 | 20-30 | 115 |
| | Clay Engineered Embankment | 50-200 | 28-30 | 115 |
| | Clay Non-engineered Embankment | 50-100 | 22-26 | 115 |
| Silt | | 0 | 28-32 | 120 |
| Clayey Sand and Silty Sand | | 0 | 28-33 | 125 |
| Sand | | 0 | 30-35 | 130 |
| Gravel and Drain Rock | | 0 | 35-40 | 135 |

### 11.4 SEEPAGE AND STABILITY ANALYSIS RESULTS

The following section presents the results of geotechnical steady state seepage and slope stability analyses, in accordance with the methodology described in the Section 11.1 through 11.3. The analyses cross-sections were evaluated in accordance with design criteria described in Section 7, for water surface elevations ranging from the 25 year flood frequency to the levee crest elevation, as shown in Section 10. The analyses for each location was first performed for the without project conditions as described in Section 1.6, essentially accounting for the constructed and/or authorized levee configuration, and, if the without project conditions analyses did not meet criteria, improvements were incorporated into the analyses cross-section until criteria was met (with project conditions as described in Section 1.7). The levee improvements analyzed in this section of the report are discussed in greater detail in Section 15 in context with recommendations to address other failure modes.

Enclosure 2 contains compiled tables of hydraulic conductivities and material strength parameters assigned for each cross-section used in analysis. Enclosure 3 contains a tabulation of the complete analyses results (seepage gradients and slope stability factors of safety for various WSE). Plates of cross-section geometry, stratigraphy, total head contours (seepage analysis) and failure surfaces (slope stability analysis) for the 200 year water surface elevation are included in the enclosure.

The following sections present the analyses results for without and with project conditions at each of the cross-section locations. Figures presented for each cross-section display underseepage average vertical exit gradient calculated at the landside levee toe and slope stability factor of safety for the analyzed water surface elevations.

#### 11.4.1 NORTH BASIN – PORT NORTH LEVEE – STA. 117+37

The without project conditions seepage and landside slope stability analysis of the Port North Levee Sta. 117+37 met both gradient and stability criteria for all water surface elevations analyzed. The freeboard criteria, corresponding to the 200yr WSE plus 3 ft (23.9 ft NAVD88), was not met. The with project condition analyzed a saddled embankment raise of select levee fill with a keyway on the landside to an elevation of 23.9 ft NAVD88. The with project conditions seepage and landside slope stability analysis met both gradient and stability criteria for all water surface elevations analyzed. Figure 11-3 displays the without project conditions analyses results and Figure 11-4 displays the with project analyses results for analyzed flood frequencies.

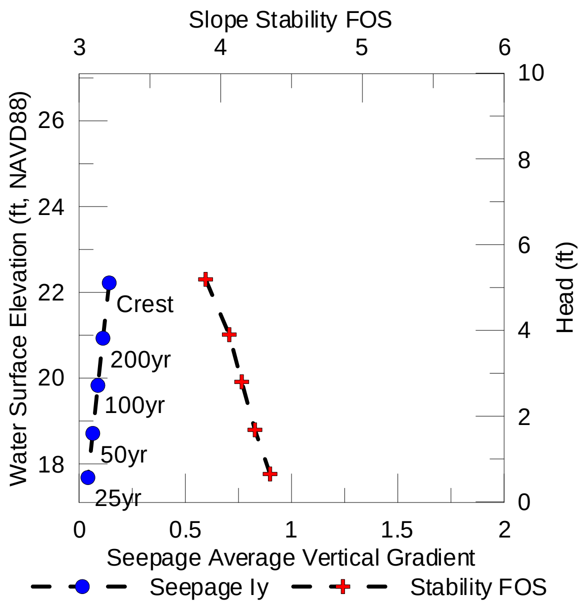



**Figure 11-3: North Basin – Port North Levee – Sta. 117+37 - Without Project Analyses Results**

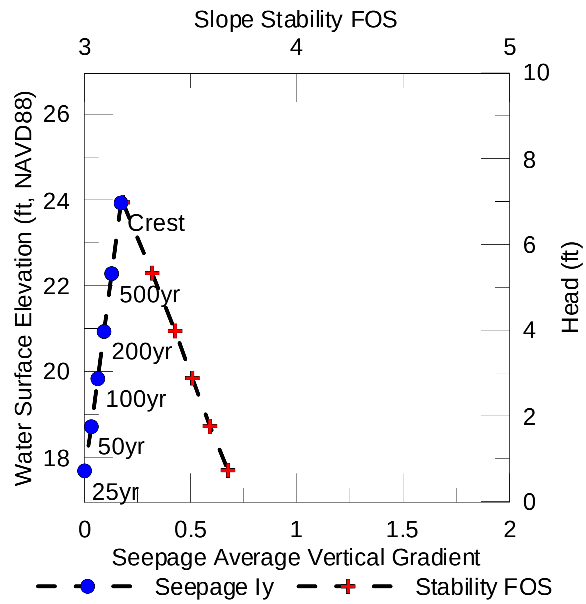



**Figure 11-4: North Basin – Port North Levee – Sta. 117+37 - With Project Analyses Results**

### 11.4.2 NORTH BASIN – SACRAMENTO BYPASS SOUTH LEVEE – STA. 32+00

The without project conditions seepage analysis of the Sacramento Bypass South Levee Sta. 32+00 have shown the potential for seepage gradients to exceed criteria beginning at the 25 yr flood frequency event due to shallow leaky silty sand (SM) layer at the levee base as well as a directly charged poorly graded silty sand (SP & SP-SM) and silty sand (SM) aquifer. Without project conditions landside stability analysis met criteria for all water surface elevations analyzed. The 25 yr flood frequency event corresponds to a water surface elevation of 31.42 ft and 16.24 ft of head on the levee embankment.

The with project conditions analyses addressed the underseepage deficiencies by incorporating a cutoff wall keyed-in to a low permeability confining layer at elevation -40.0 ft. With the improvement measures described above, the seepage and stability analyses met criteria at all flood frequencies. Figure 11-5 displays the without project conditions analyses results and Figure 11-6 displays the with project analyses results for analyzed flood frequencies.

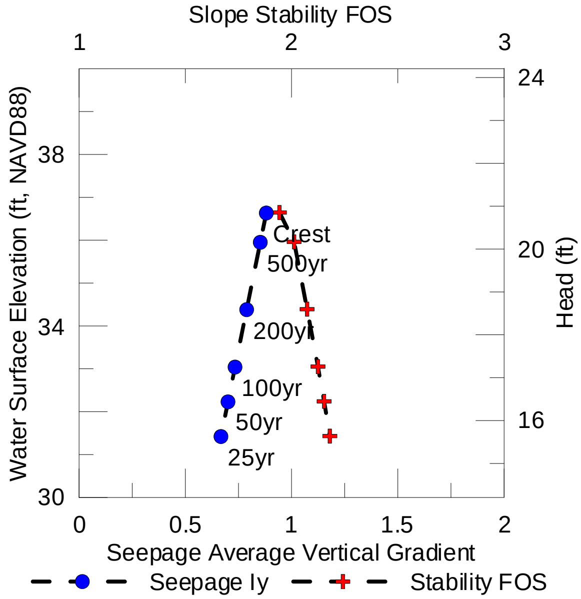



**Figure 11-5: North Basin – Sacramento Bypass South Levee – Sta. 32+00 - Without Project Analyses Results**

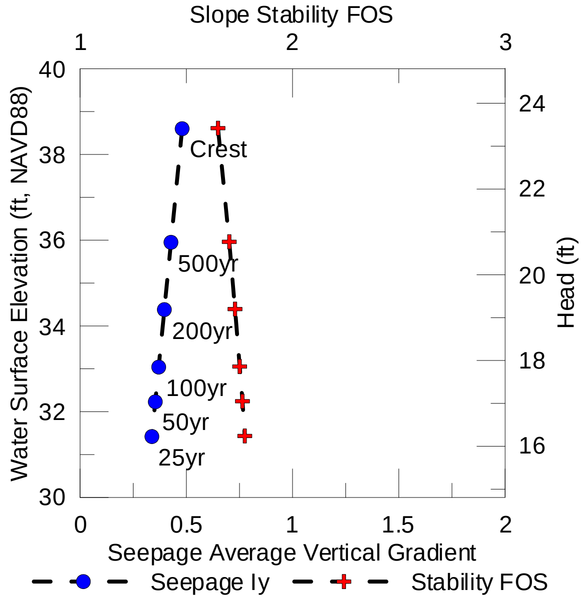



**Figure 11-6: North Basin – Sacramento Bypass South Levee – Sta. 32+00 - With Project Analyses Results**

### 11.4.3 NORTH BASIN – SACRAMENTO BYPASS SOUTH LEVEE – STA. 52+00

The without project conditions seepage analysis of the Sacramento Bypass South Levee Sta. 52+00 met criteria for all water surface elevations analyzed. Stability analyses showed the potential for landside slope instability with water surfaces near the crest of the embankment. Subsurface conditions and landside slopes are analogous to the analysis section at Sta. 32+00. Sacramento Bypass South Levee Sta. 52+00 was completed as part of the West Sacramento Levee System F3 Geotechnical Reevaluation Report – June 2011. The F3 report focused on locating deficiencies; as such, the report did not analyze mitigation measures. Figure 11-7 displays the without project conditions analyses results. Following review of subsurface conditions and past performance in the reach, the anticipated remedial improvement measure prescribed is a shallow cutoff wall constructed to elevation 5ft (NAVD 88).

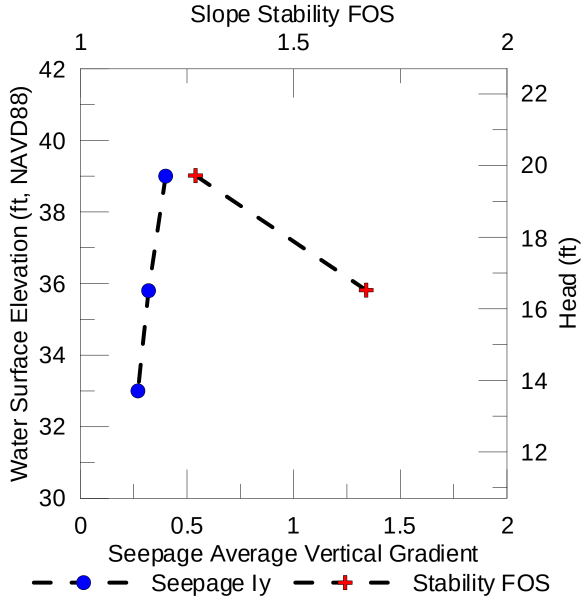



**Figure 11-7: North Basin – Sacramento Bypass South Levee – Sta. 52+00 - Without Project Analyses Results**

### 11.4.4 NORTH BASIN – SACRAMENTO RIVER WEST LEVEE – STA. 96+00

The without project conditions seepage analyses of the Sacramento River West Levee Sta. 96+00 have shown the potential for seepage gradients to exceed criteria beginning at the 50 yr flood frequency event. The 50 ft thick aquifer layer of poorly graded silty sand (SP & SP-SM) and poorly graded sands with gravels (SP) is directly charged which contributes to the underseepage issue. Without project conditions landside stability analysis did not meet criteria for all water surfaces analyzed beginning at the 25 yr flood frequency. In comparison to past performance, there was no mention detailing a slope stability concern. However, the potential for an underseepage driven slope stability failure may exist for this location. The 50 yr flood frequency event corresponds to a water surface elevation of 34.03 ft and 11.62 ft of head and the 25 yr flood frequency event corresponds to a water surface elevation of 33.49 ft and 11.08 ft of head on the levee embankment.

The with project conditions analyses addressed the underseepage and landside slope stability deficiencies by incorporating a cutoff wall keyed-in to a low permeability confining layer at elevation -65.0 ft. The with project conditions analyses evaluated the recommendation contained in the Rivers Early Implementation Program (EIP). With the improvement measures described above, the seepage and stability analyses met criteria at all flood frequencies. Figure 11-8 displays the without project conditions analyses results and Figure 11-9 displays the with project analyses results for analyzed flood frequencies.

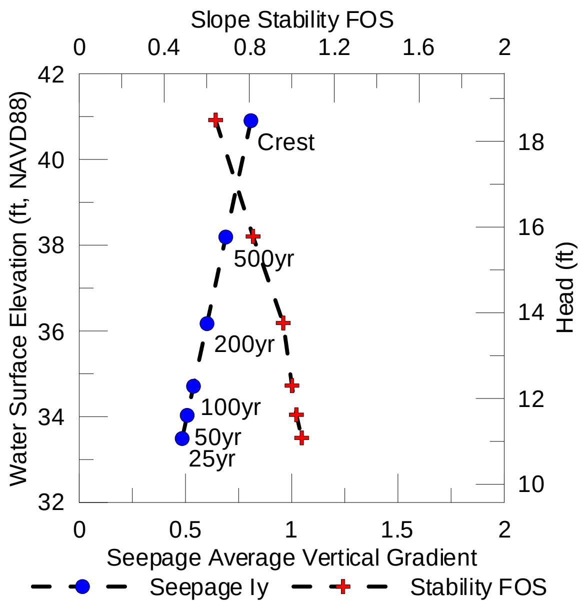



**Figure 11-8: North Basin – Sacramento River West Levee – Sta. 96+00 - Without Project Analyses Results**

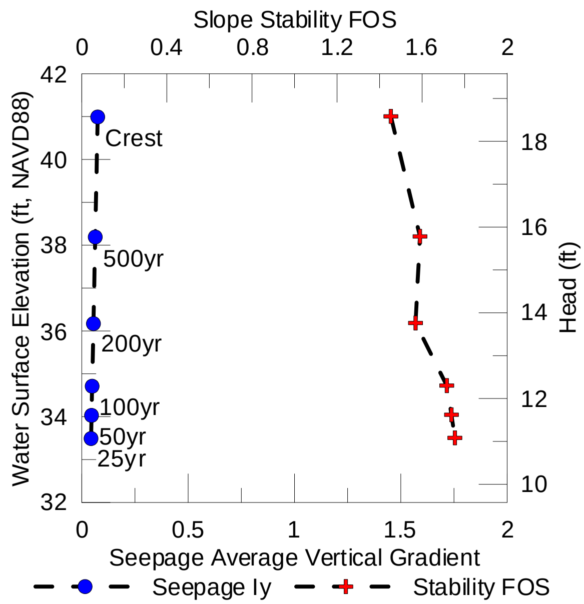



**Figure 11-9: North Basin – Sacramento River West Levee – Sta. 96+00 - With Project Analyses Results**

### 11.4.5 NORTH BASIN – SACRAMENTO RIVER WEST LEVEE – STA. 190+00

The without project conditions seepage analysis of the Sacramento River West Levee Sta. 190+00 met gradient criteria for all water surface elevations analyzed. Without project conditions landside stability analysis did not meet criteria for all water surfaces analyzed beginning at the 25 yr flood frequency. The slope stability issue can be attributed to an oversteepened landside slope and high plasticity clays in the levee embankment. The 25 yr flood frequency event corresponds to a water surface elevation of 33.36 ft and 5.36 ft of head on the levee embankment.

The with project conditions analyses addressed the landside slope stability deficiencies by incorporating a cutoff wall keyed-in to a low permeability confining layer at elevation -65.0 ft and flattening of the landside slope to a minimum of 3H:1V. While the without project conditions show the criteria for seepage being met, the recommendation of a keyed-in cutoff wall would provide continuity to adjoining project reaches as well as mitigate against potential defects in the blanket layer. The construction of the cutoff wall would also address the aquifer layer as a whole. With the improvement measures described above, the seepage and stability analyses met criteria at all flood frequencies. Figure 11-10 displays the without project conditions analyses results and Figure 11-11 displays the with project analyses results for analyzed flood frequencies.

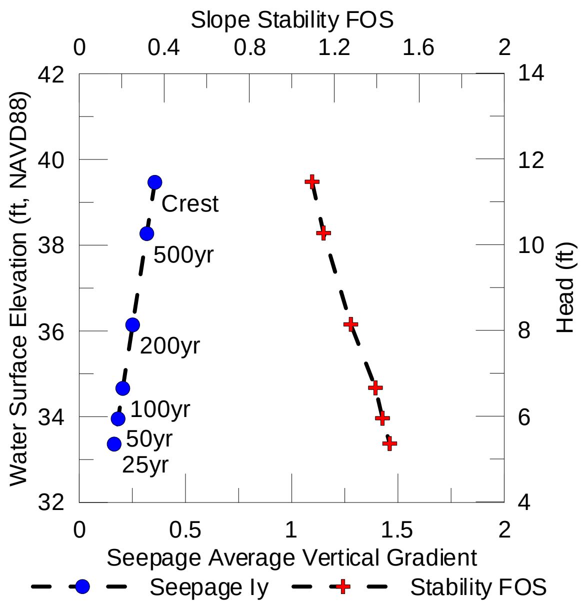



**Figure 11-10: North Basin – Sacramento River West Levee – Sta. 190+00 - Without Project Analyses Results**

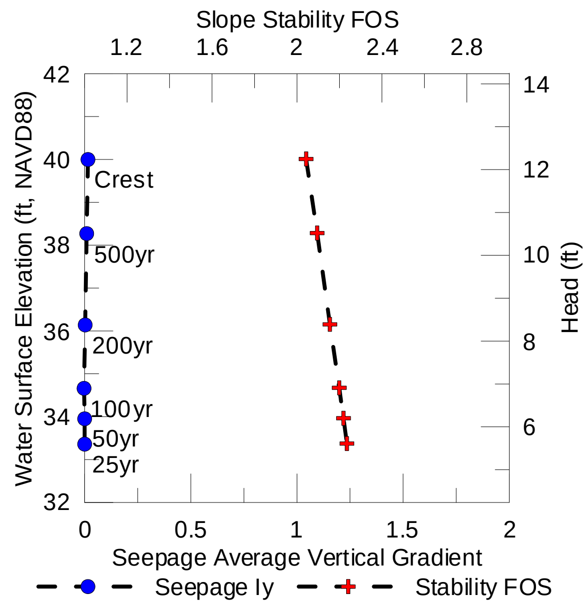



**Figure 11-11: North Basin – Sacramento River West Levee – Sta. 190+00 - With Project Analyses Results**

### 11.4.6 NORTH BASIN –YOLO BYPASS EAST LEVEE– STA. 36+00

The without project conditions seepage analysis of the Yolo Bypass East Levee Sta. 36+00 did not meet gradient criteria for all water surface elevations analyzed beginning at the 25 yr flood frequency.  A shallow foundation silty sand (SM) layer at the base of the embankment coupled with a directly charged deeper aquifer comprised of a poorly graded silty sand (SP-SM) contribute to the seepage deficiency.  Without project conditions landside stability analysis met criteria for all water surfaces analyzed.  The 25 yr flood frequency event corresponds to a water surface elevation of 29.41 ft and 10.05 ft of head on the levee embankment.

When relating the past performance of this area to the analysis results, a discrepancy can be noted.  This can be attributed to construction actions which placed and compacted clay fill over the existing levee embankment, which is accounted for in the without project conditions.  This construction followed the flood events of 1997 and was completed between 1998 and 2002.  The placement of compacted clay fill may have mitigated a potential landside slope instability problem, but did not address the potential shallow underseepage deficiency of the silty sand layer.

The with project conditions analyses addressed both shallow and deep underseepage deficiencies by incorporating a cutoff wall keyed-in to a low permeability confining layer at elevation -10.0 ft and flattening of the landside slope to a minimum of 3H:1V.  With the improvement measures described above, the seepage and stability analyses met criteria at all flood frequencies.  Figure 11-12 displays the without project conditions analyses results and Figure 11-13 displays the with project analyses results for analyzed flood frequencies.

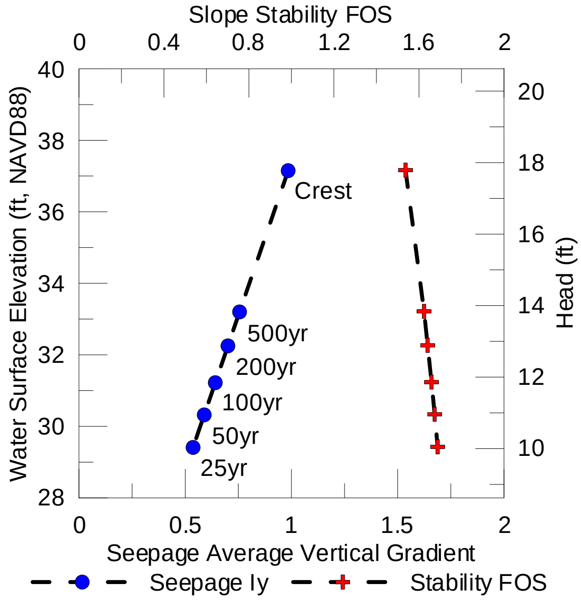



**Figure 11-12: North Basin – Yolo Bypass East Levee – Sta. 36+00 - Without Project Analyses Results**

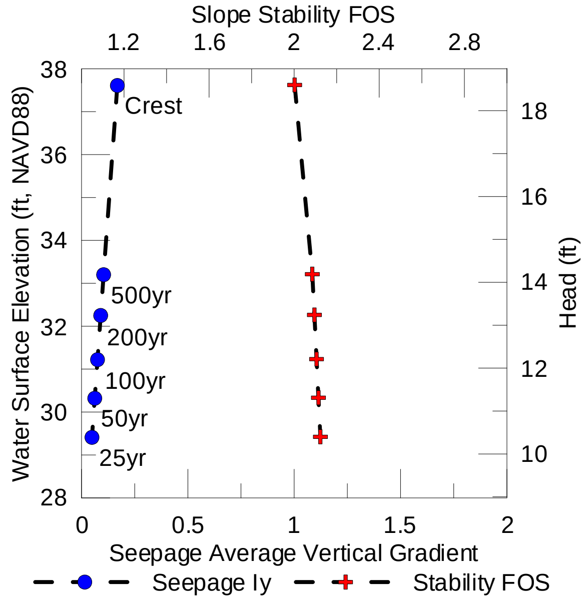



**Figure 11-13: North Basin – Yolo Bypass East Levee – Sta. 36+00 - With Project Analyses Results**

### 11.4.7 NORTH BASIN –YOLO BYPASS EAST LEVEE– STA. 107+31

The without project conditions seepage and landside slope stability analysis of the Yolo Bypass East Levee Sta. 107+31 did not meet gradient criteria for all water surface elevations analyzed. The cases analyzed for Yolo Bypass East Levee Sta. 107+31 were contained within the West Sacramento Levee System F3 Geotechnical Reevaluation Report – June 2011. The F3 report focused on locating deficiencies; as such, the report did not analyze mitigation measures under Contract C (Sta. 104+73 to 118+50) which was not finalized at the time of the analysis. The results identified both a seepage and stability deficiency. Figure 11-14 displays the without project conditions analyses results.

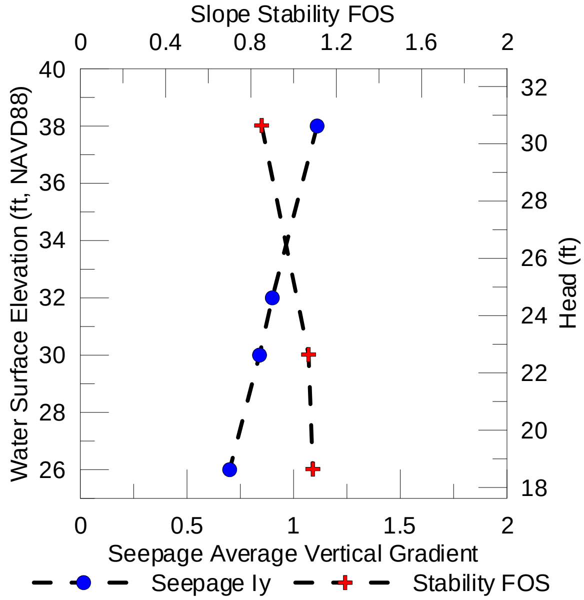



**Figure 11-14: North Basin – Yolo Bypass East Levee – Sta. 107+31 - Without Project Analyses Results**

### 11.4.8 SOUTH BASIN –DEEP WATER SHIP CHANNEL WEST LEVEE– STA. 12+00

The without project conditions seepage analysis of the Deep Water Ship Channel West Levee Sta. 12+00 did not meet gradient criteria for all water surface elevations analyzed beginning at the 25 yr flood frequency. A directly charged deeper aquifer comprised of a poorly graded silty sand (SP-SM) and poorly graded sand (SP) contributed to the underseepage deficiency. Without project conditions landside stability analysis met criteria for all water surfaces analyzed beginning at the 25 yr flood frequency as the existing embankment slopes are greater than 4H:1V. The 25 yr flood frequency event corresponds to a water surface elevation of 17.68 ft and 15.18 ft of head on the levee embankment. The DWSC West Levee, while notable in length of 21 miles, the analysis section characterizes approximately 25% of the reach length where the critical geometry and soil conditions exist within the northern most portion of the reach. The location of the analysis section is at the most critical from a levee height and net head on the embankment perspective. Moving further downstream for the remainder of the project reach, there are no recommended mitigation measures as the embankment geometry widens, the embankment slopes are flattened, and the net head on the embankment is decreased.

The with project conditions analyses addressed underseepage deficiencies by incorporating a cutoff wall keyed-in to a low permeability confining layer at elevation -60.0 ft. With the improvement measures described above, the seepage and stability analyses met criteria at all flood frequencies. Figure 11-15 displays the without project conditions analyses results and Figure 11-16 displays the with project analyses results for analyzed flood frequencies.

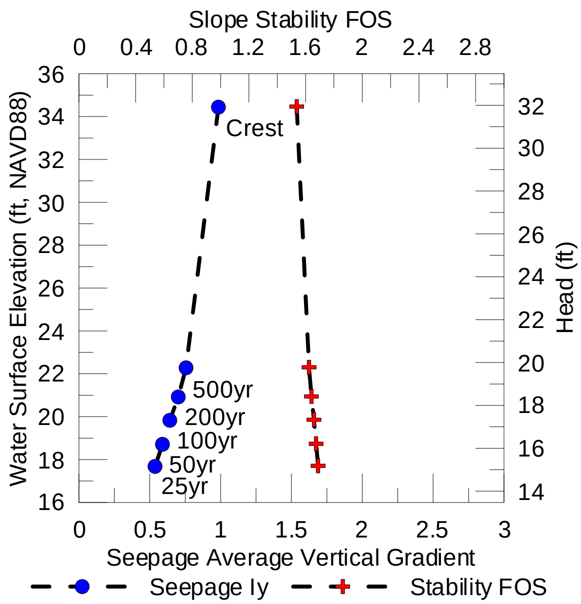



**Figure 11-15: South Basin – Deep Water Ship Channel West Levee– Sta. 12+00 - Without Project Analyses Results**

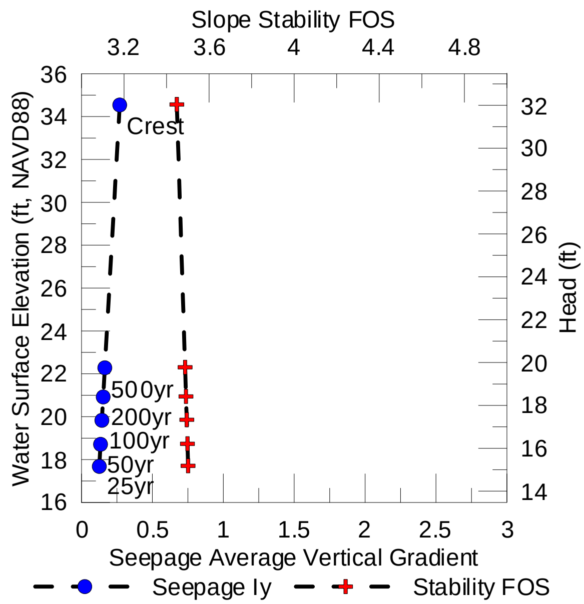



**Figure 11-16: South Basin – Deep Water Ship Channel West Levee– Sta. 12+00 - With Project Analyses Results**

### 11.4.9 SOUTH BASIN – PORT SOUTH LEVEE – STA. 123+55

The without project conditions seepage and landside slope stability analysis of the Port South Levee Sta. 123+55 met both gradient and stability criteria for all water surface elevations analyzed. The freeboard criteria, corresponding to the 200yr WSE plus 3 ft (23.93 ft NAVD88), was not met. The with project condition analyzed an embankment raise of select levee fill to an elevation of 23.93 ft NAVD88.

The with project conditions seepage and landside slope stability analysis met both gradient and stability criteria for all water surface elevations analyzed and also incorporated a cutoff wall keyed into a low permeability layer at elevation -53.0ft to address potential variations in the blanket materials that may lead to the development of preferential seepage paths. The recommended mitigation of an underseepage cutoff wall addresses the historic seepage concerns inherent to the adjacent area. From Sta. 120+00 to Sta. 130+00, along the landside of the levee embankment the basin of historic Lake Washington exists. The former lake bed contains basin and channel deposits beneath the foundation of the present day embankment which are susceptible to underseepage. Inclusion of a cutoff wall in this location would mitigate against this potential. Figure 11-17 displays the without project conditions analyses results and Figure 11-18 displays the with project analyses results for analyzed flood frequencies.

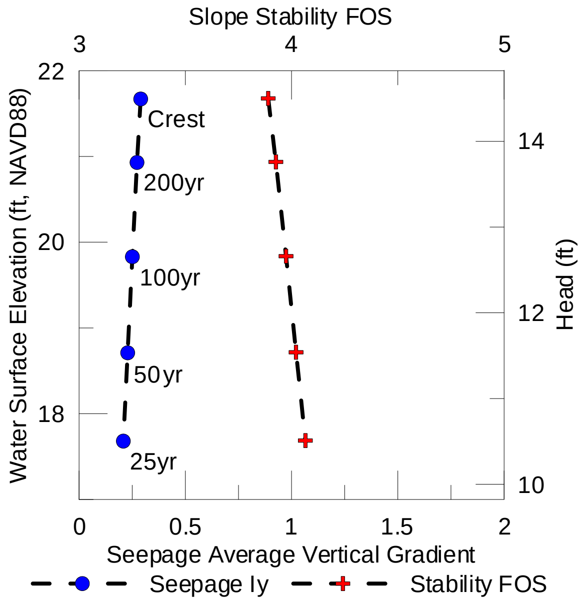



**Figure 11-17: South Basin – Port South Levee – Sta. 123+55 - Without Project Analyses Results**

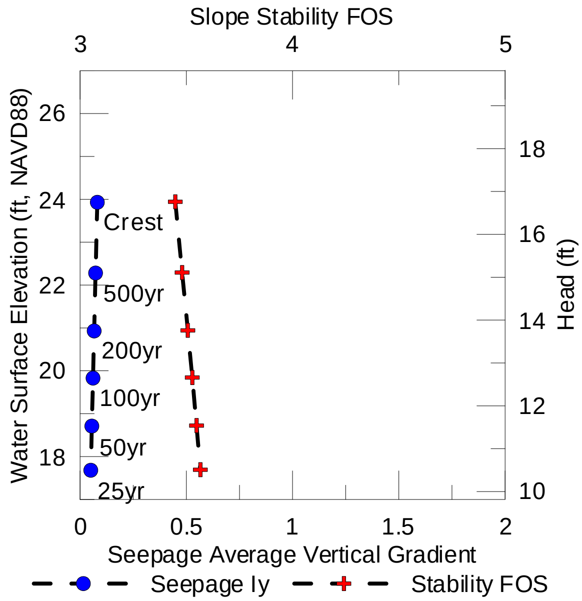



**Figure 11-18: South Basin – Port South Levee – Sta. 123+55 - With Project Analyses Results**

### 11.4.10 SOUTH BASIN–SOUTH CROSS LEVEE– STA. 17+50

The without project conditions seepage and landside slope stability analysis of the South Cross Levee Sta. 17+50 did not meet both gradient and stability criteria for all water surface elevations analyzed. This coincides with the past performance issues noted during the seepage events of 1963 and 1965. The freeboard criteria, corresponding to the 200yr WSE plus 3 ft (32.29 ft NAVD88), was not met. The with project condition analyzed an embankment raise of select levee fill to an elevation of 35.29 ft NAVD88.

The with project conditions seepage and landside slope stability analysis met both gradient and stability criteria for all water surface elevations analyzed by incorporating landside relief wells spaced parallel to the levee alignment at 50 ft spacing to a depth of 70 ft. The 70 ft well depth will include 2 screened intervals from an elevation of -9.5 to -23.5 ft and from -39.5 to -58 ft NAVD88. Further detail of the calculations is provided in Appendix 9. The analysis results showed that with a loading to the top of the levee embankment, the uplift gradient criteria was met at a well spacing of 50ft. Figure 11-19 displays the without project conditions analyses results. With project results incorporating relief well analysis will contain calculations for total flow and well spacing; current software constraints do not allow for steady state seepage and landside stability analysis using FEM. Further detail to the relief well design will be included in feasibility level design documentation.

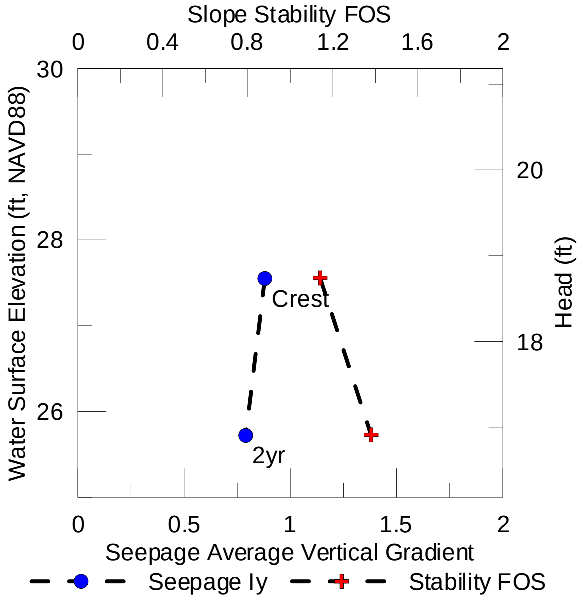



**Figure 11-19: South Basin –South Cross Levee – Sta. 17+50 - Without Project Analyses Results**

### 11.4.11 SOUTH BASIN –SACRAMENTO RIVER WEST LEVEE– STA. 264+00

The without project conditions seepage and landside slope stability analysis of the Sacramento River West Levee Sta. 264+00 did not meet either gradient and stability criteria for all water surface elevations analyzed beginning at the 25 yr flood frequency. The 25 yr flood frequency event corresponds to a water surface elevation of 31.83 ft and 12.21 ft of head on the levee embankment. Primarily, the existing levee embankment and upper foundation is comprised of poorly graded sand (SP) and poorly graded silty sand which contribute to a shallow underseepage and through seepage issues.

The with project conditions analyses addressed seepage and landside slope stability deficiencies by incorporating a hanging cutoff wall to elevation -5.0 ft and placement of a 80 ft wide drained seepage berm. While the analysis at this location shows a hanging cutoff wall the analysis section represents the critical cases for the project reach. It should be noted that throughout the reach there maybe portions of hanging cutoff wall as well as keyed-in portions to a low permeability confining layer. Figure 11-20 displays the without project conditions analyses results and Figure 11-21 displays the with project analyses results for analyzed flood frequencies.

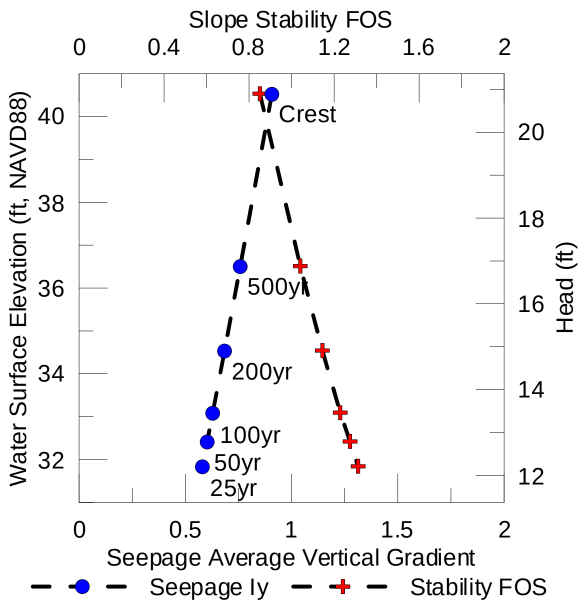



**Figure 11-20: South Basin – Sacramento River West Levee – Sta. 264+00 - Without Project Analyses Results**

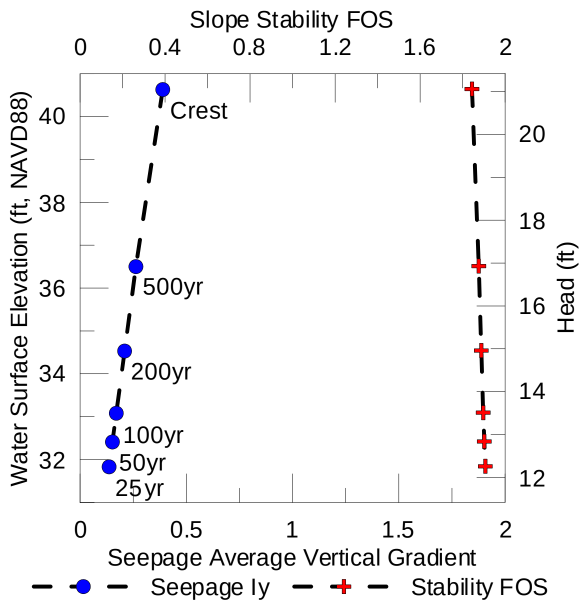



**Figure 11-21: South Basin – Sacramento River West Levee – Sta. 264+00 - With Project Analyses Results**

### 11.4.12 SOUTH BASIN –SACRAMENTO RIVER WEST LEVEE– STA. 80+00

The without project conditions seepage and stability analysis of the Sacramento River West Levee Sta. 80+00 met gradient for all water surface elevations analyzed. Primarily, the existing levee embankment and upper foundation are comprised of poorly graded sand (SP) and poorly graded silty sand which contributes to shallow underseepage and through seepage issues. Both the levee embankment and upper foundation materials are directly charged from the channel further contributing to potential distresses.

The with project conditions analyses addressed through seepage deficiencies and shallow underseepage concerns by incorporating a cutoff wall keyed-in to a low permeability confining layer at elevation -5.0 ft and placement of a 80 ft wide drained seepage berm. Figure 11-22 displays the without project conditions analyses results and Figure 11-23 displays the with project analyses results for analyzed flood frequencies.

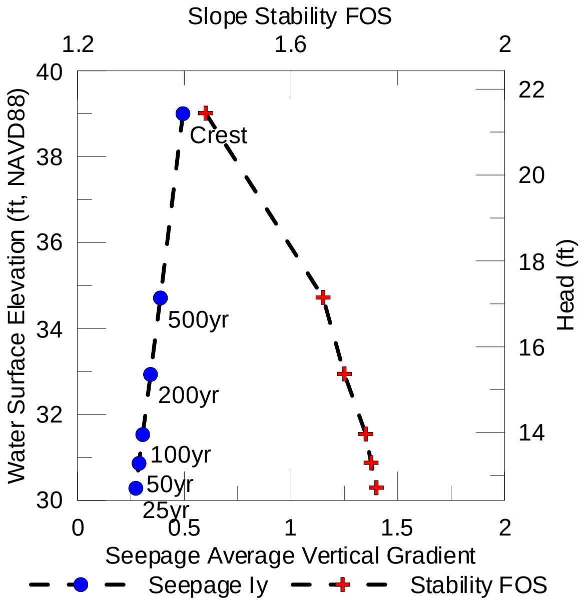



**Figure 11-22: South Basin – Sacramento River West Levee – Sta. 80+00 - Without Project Analyses Results**

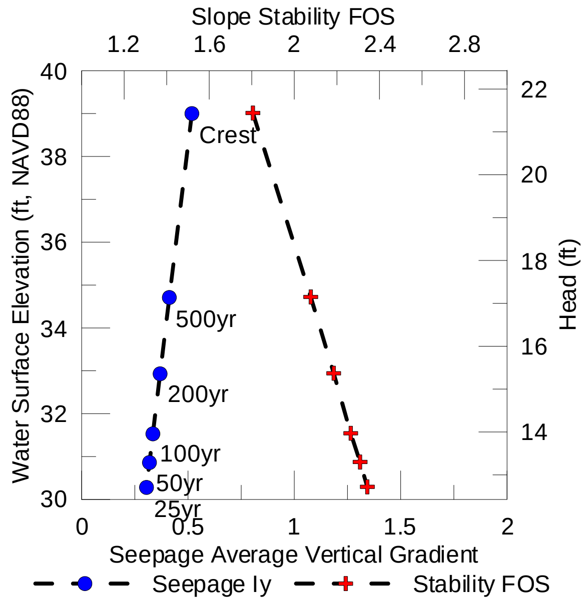



**Figure 11-23: South Basin – Sacramento River West Levee – Sta. 80+00 - With Project Analyses Results**

### 11.4.13 SOUTH BASIN –SACRAMENTO RIVER WEST LEVEE– STA. 35+22

In this project reach the SUALRP constructed a shallow through seepage cutoff wall in the early 1990s; subsequent flood events resulted in boils and seepage distresses in both 1995 and 1998. The without project conditions analysis did not correlate to past performance. The through seepage cutoff wall was not included in the analysis section as the past performance events resulting in seepage distress leads way to the overall functionality of the wall itself. The without project conditions seepage and stability analysis of the Sacramento River West Levee Sta. 35+22 met gradient and factor of safety requirements for all water surface elevations analyzed. Primarily, the existing levee embankment and upper foundation are comprised of silts and silty sands (ML and SM) and sands interbedding the clay and silt foundation layers respectively.

The with project conditions analyses addressed the potential deficiencies by incorporating a keyed-in cutoff wall to tip elevation -5.0 feet which would mitigate the interbedding of the upper foundation and allow for excess uplift gradient pressures to be relieved. Figure 11-24 displays the without project conditions analyses results and Figure 11-25 displays the with project analyses results for analyzed flood frequencies.

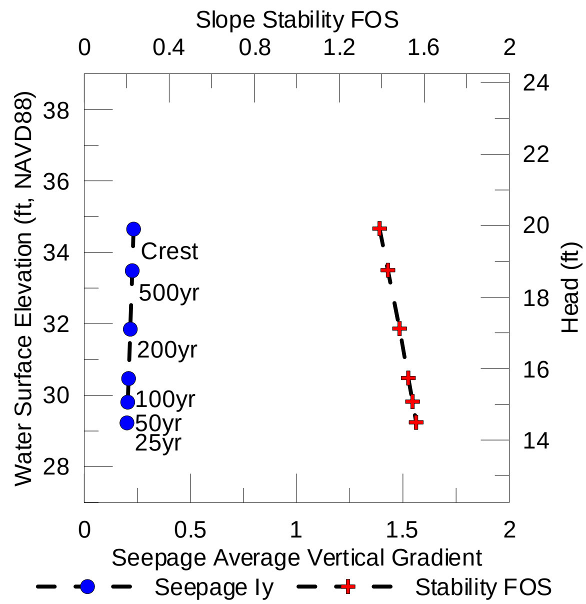



**Figure 11-24: South Basin – Sacramento River West Levee – Sta. 35+22 - Without Project Analyses Results**

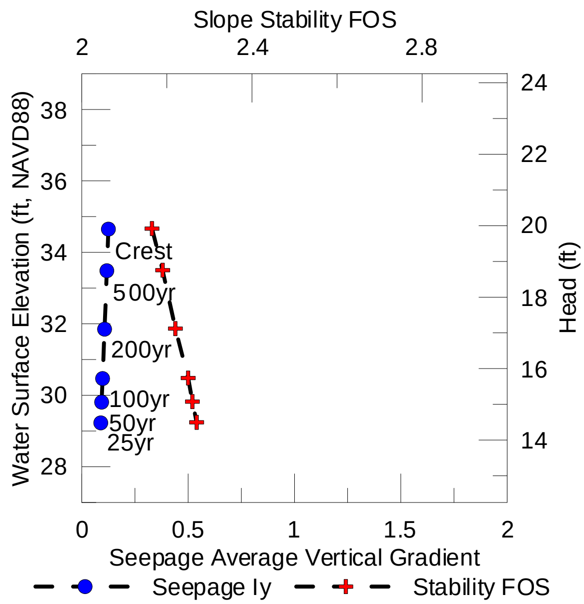



**Figure 11-25: South Basin – Sacramento River West Levee – Sta. 35+22 - With Project Analyses Results**

### 11.4.14 SOUTH BASIN –YOLO BYPASS EAST LEVEE– STA. 10+00

The without project conditions seepage and landside slope stability analysis of the Yolo Bypass East Levee Sta. 10+00 met gradient and stability criteria for all water surface elevations analyzed. The freeboard criteria, corresponding to the 200yr WSE plus 3 ft (34.93 ft NAVD88), was not met. The with project condition analyzed an embankment raise of select levee fill to an elevation of 34.93 ft NAVD88.

The with project conditions seepage and landside slope stability analysis met both gradient and stability criteria, as well as satisfied the freeboard height requirement. For all water surface elevations analyzed, a cutoff wall keyed into a low permeability layer was included at elevation -60.0 ft to address potential variations in the blanket materials that and foundation layers that may lead to deep underseepage issues as the channel directly charges the foundation layers. Figure 11-26 displays the without project conditions analyses results and Figure 11-27 displays the with project analyses results for analyzed flood frequencies.

The feasibility analysis at the Sacramento River north levee does not demonstrate the need for seepage or stability mitigation. Several other reports prepared for WSAFCA by others indicated the need for seepage or stability mitigation modifications. Based on the information available at the feasibility level, and the conflict between recommendations from the sponsor and the Corps, the geotechnical recommendation was to recommend work in this area, with the final determination of need to be made during PED.

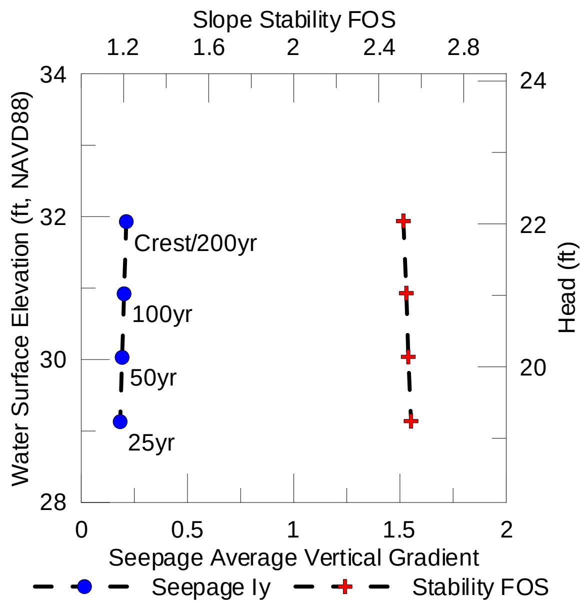



**Figure 11-26: South Basin – Yolo Bypass East Levee – Sta. 10+00 - Without Project Analyses Results**

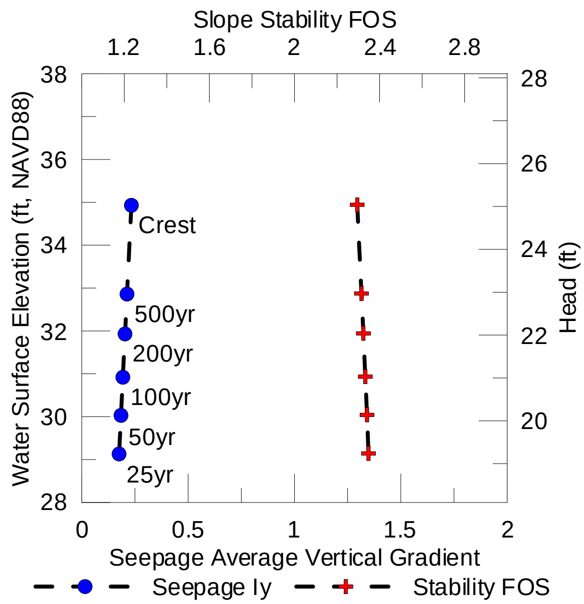



**Figure 11-27: South Basin – Yolo Bypass East Levee – Sta. 10+00 - With Project Analyses Results**

### 11.4.15 SOUTH BASIN – YOLO BYPASS EAST LEVEE – STA. 53+96

The without project conditions seepage and landside slope stability analysis of the Yolo Bypass Levee Sta. 53+96 did not meet either gradient and stability criteria for all water surface elevations analyzed beginning at the 25 yr flood frequency. The gradients and factors of safety incorporated a ditch at landside levee toe; a ditch empty case was analyzed. The 25 yr flood frequency event corresponds to a water surface elevation of 27.53 ft and 27.10 ft of head on the levee embankment. The amount of differential head on the levee embankment coupled with the foundation materials being directly charged by the channel and a thick poorly graded sand layer, each contribute to the seepage and slope stability deficiencies. The freeboard criteria, corresponding to the 200yr WSE plus 3 ft (33.26 ft NAVD88), was not met. The with project condition analyzed an embankment raise of select levee fill to an elevation of 33.26 ft NAVD88.

The with project conditions analyses addressed the seepage and landside slope stability deficiencies by incorporating an 80 ft wide drained seepage berm at the landside levee toe, slope flattening to a minimum of 3.0H:1.0V, and an embankment raise to satisfy freeboard requirements. Figure 11-28 displays the without project conditions analyses results and Figure 11-29 displays the with project analyses results for analyzed flood frequencies.

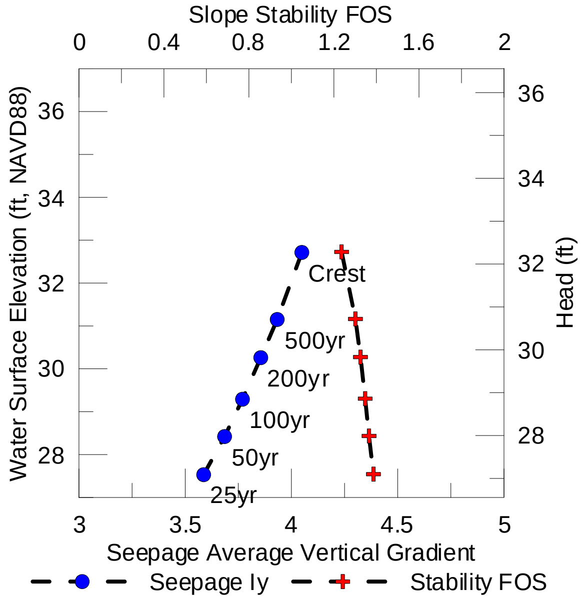



**Figure 11-28: South Basin – Yolo Bypass East Levee – Sta. 53+96 - Without Project Analyses Results**

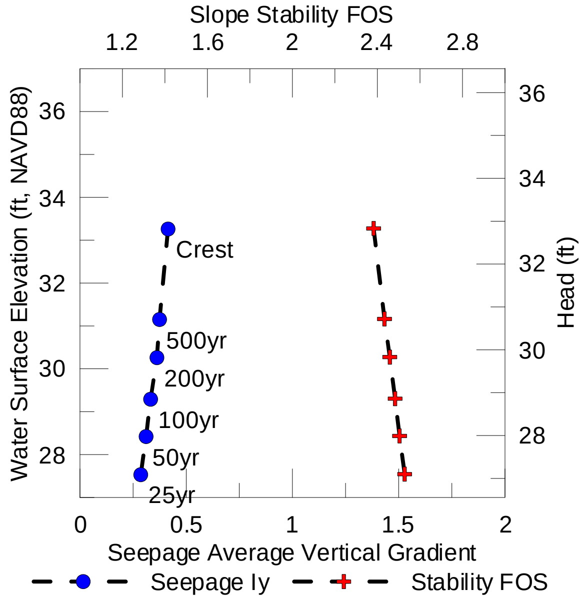



**Figure 11-29: South Basin – Yolo Bypass East Levee – Sta. 53+96 - With Project Analyses Results**

## 11.5 SACRAMENTO BYPASS NORTH LEVEE

As the Sacramento Bypass North levee is located to the north of the north project basin; a separate discussion of the results is provided irrespective of the project basins.

### 11.5.1 NORTH BASIN –SACRAMENTO BYPASS NORTH LEVEE– STA. 8+30

The without project conditions seepage analysis of the Sacramento Bypass North Levee Sta. 8+30 met both gradient for all water surface elevations analyzed. The freeboard criteria, corresponding to the 200yr WSE plus 3 ft (36.36 ft NAVD88), was not met. The with project condition analyzed an embankment raise of select levee fill to an elevation of 36.36 ft NAVD88. Slope stability criteria was not met for all water surface elevations were not met

The with project conditions analyses addressed landside slope stability deficiencies by incorporating an 80 ft wide drained berm at the landside levee toe, and an embankment raise was also included to satisfy freeboard requirements. Figure 11-30 displays the without project conditions analyses results and Figure 11-31 displays the with project analyses results for analyzed flood frequencies.

Analyses was performed on the existing Sacramento Bypass north levee to determine the performance of that levee as well as the general material composition of the levee. However, the project alternative recommendation was for a relocated north levee. There are no existing borings or other geotechnical data available for the location of the new north Sacramento bypass levee. Therefore, a conservative assumption was made regarding potential seepage or stability improvements required for the new north levee. These assumptions were intended to reasonably maximize real estate and environmental impacts for the planning study, with final determination of need to be determined in PED.

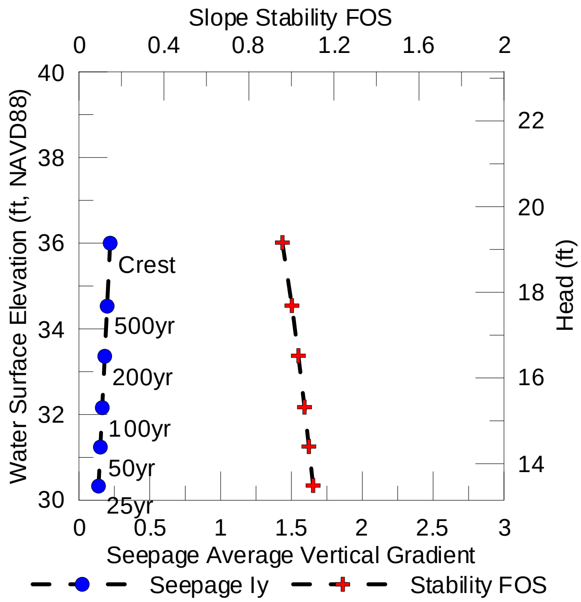



**Figure 11-30: Sacramento Bypass North Levee – Sta. 8+30 - Without Project Analyses Results**

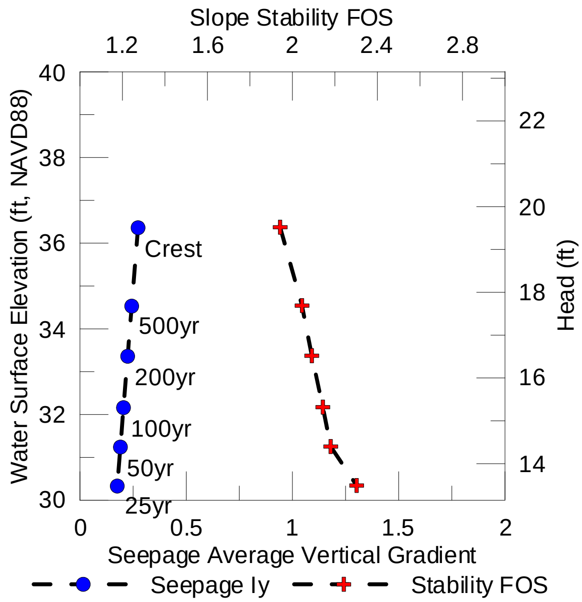



**Figure 11-31: Sacramento Bypass North Levee – Sta. 8+30 - With Project Analyses Results**

## 12.0 SEISMIC ASSESSMENT

To evaluate the potential to liquefaction resistance of soils, liquefaction triggering analysis was performed based on the procedure from the summary report of the 1996 National Center for Earthquake Engineering Research (NCEER) and 1998 NCEER/National Science Foundation (NSF) Workshops on Evaluation of Liquefaction Resistance of Soils, published as part of the Journal of Geotechnical and Geoenvironmental Engineer, dated October 2001 (Youd, Idriss, Andrus, & Arango, October 2001). The seismic assessment is included as Enclosure 6.

Probabilistic Seismic Hazard Analysis (PSHA) based on the 2008 Next Generation Attenuation (NGA) relationships was used to develop the seismic loading parameters in this study. The deaggregations are from the United States Geologic Survey (USGS) developed 2008 Interactive Deaggregations web program. The mean magnitude or the weighted average considering the percent contribution to the total hazard for the used for the study levees is 6.60 to 6.67 dependent on location. A peak horizontal ground horizontal acceleration contour map is produced using outputs from the USGS deaggregation program for 20% exceedance in 50 years (224-year average return period). Site Class D as defined by the USGS site classification for seismic assessment was used for this study because the locations selected for evaluation contain harmonic mean $N_{60}$ blow counts ranged between 9.9 and 19.9 and a median value of 15.4. The corresponding shear wave velocity, $V_{s30}$, is 234 m/s for the study area.

The consequences of triggering liquefaction include flow slide or post earthquake instability and lateral spreading. Where static driving shear stress is greater than the resisting strengths (residual strength), a global or structural failure can occur, leading to loss of freeboard, cracking, and increased piping. Lateral deformation can also develop as a consequence of instability due to loss of shear strength or as accumulation of shear strains throughout the soil profile. Lateral spreading towards any open channel or face can occur in mildly sloping ground and extend to very large distances away from the open face. Vertical displacement can develop as a consequence of reconsolidation of the liquefied soil. For this study, global or structural stability is evaluated where liquefiable layers with factor of safety less than 1.4 is found. Lateral spreading and post-liquefaction reconsolidation settlement were considered only when structural stability had a factor of safety greater than 1.0.

Where liquefiable layers were found to have a factor of safety less than one and between 1.0 and 1.4, static limit equilibrium stability analysis using UTEXAS4 based on Spencer’s method was performed. Automatic circular shear surface search and non-circular or wedge shear surface search were performed for both the landside and waterside in UTEXAS4. Post-earthquake residual shear strength was used for the liquefiable layers. The residual strength was estimated per Olson and Stark, 2002.

The post seismic flood protection ability for each section analyzed is summarized below. The post-seismic flood protection ability is defined as the ability to assume the current or designed flood protection ability after a 200yr earthquake. Further discussion of analysis results and methodologies are contained in Enclosure 6

| NORTH BASIN | | SOUTH BASIN | |
|---|---|---|---|
| **Reach** | **Post-Seismic Flood Protection Ability** | **Reach** | **Post-Seismic Flood Protection Ability** |
| Sacramento River West North Levee | Low Vulnerability | South Cross Levee | Low Vulnerability |
| Sacramento Bypass Levee* | Medium Vulnerability | Deep Water Ship Channel East Levee | Low Vulnerability |
| Yolo Bypass Levee* | Low Vulnerability | Deep Water Ship Channel West Levee | Low Vulnerability |
| Port North Levee | Low Vulnerability | Port South Levee | Low Vulnerability |
| | | Sacramento River West South Levee** | High Vulnerability |

*No water behind the levee during non-flood season.

## 13.0 GEOTECHNICAL RECOMMENDATIONS FOR LEVEE IMPROVEMENTS

As presented in previous sections of this report, the levees protecting the West Sacramento study area are susceptible to through seepage, underseepage, slope stability, and erosion. In some locations, on the levees along the West Sacramento study area, early implementation projects have been constructed and/or are in design by local stakeholders. However, deficiencies still remain throughout the project area. This section presents methods for addressing the geotechnical deficiencies that remain for the levees within the West Sacramento study area.

To address seepage and seepage related slope stability deficiencies the predominant recommendation is cutoff walls in conjunction with seepage berms where applicable, particularly considering the urban development close to the levee embankment. In other locations not necessarily as prevalent as the cutoff wall fixes relief wells, drained stability berms, and landside slope flattening were recommended. To further detail cutoff wall depth to account for variation in elevations of confining key-in layers, a review of existing subsurface information through available plan and subsurface profiles was completed. The resulting tables in the subsequent section account for this as well as coincide with deterministic analysis results.

Based on hydraulic modeling, various locations did not meet the freeboard requirement and the embankment will be raised placing fill.

In addition to geotechnical seepage and slope stability improvement recommendations to assure levee integrity; existing irrigation and drainage ditches landside of the levee would need to be relocated to a distance where there is no adverse impact on levee performance (minimum 50 feet), penetrations through the levee would be relocated and/or modified in conformance with the USACE levee safety policy, and vegetation would be managed in accordance with Section 8.5 of this report.

The following sections will detail the geotechnical recommendation and extent of their locations throughout the project area.

### 13.1 NORTH BASIN

Within the north basin of the project, the predominant recommended fix is a cutoff wall. Although the tip elevation, nature of the key-in material, and method of construction may differ, overall the main component remains the cutoff wall. The tables below detail the extent and various combinations of the geotechnical recommendations per channel.

**Table 13-1 – Geotechnical Recommendations - Sacramento Bypass South Levee**

| Sacramento Bypass - South Levee | | | | |
|---|---|---|---|---|
| Station | | | Levee | Recommended Improvements |
| From | - | To | | |
| 0+00 | - | 18+00 | In Place | None |
| 18+00 | - | 40+00 | In Place | Cutoff Wall to Elev. -40 ft (65 ft Deep) |
| 40+00 | - | 64+50 | In Place | Cutoff Wall to Elev. 5 ft (20 ft Deep) |
| 64+50 | - | 64+80 | In Place | None |

**Table 13-2 – Geotechnical Recommendations – Sacramento River West Levee**

| Sacramento River North - West Levee | | | | |
|---|---|---|---|---|
| Station | | | Levee | Recommended Improvements |
| From | - | To | | |
| 0+00 | - | 71+50 | In Place | Cutoff Wall to Elev. 0 ft (30 ft Deep) |
| 71+50 | - | 101+00 | In Place | None |
| 101+00 | - | 140+30 | In Place | Cutoff Wall to Elev. 0 ft (30 ft Deep) |
| 140+30 | - | 155+00 | In Place | Cutoff Wall to Elev. -50 ft (80 ft Deep) |
| 155+00 | - | 185+30 | In Place | Cutoff Wall to Elev. -80 ft (110 ft Deep) |
| 185+30 | - | 194+60 | In Place | Cutoff Wall to Elev. -80 ft (110 ft Deep) |
| 194+60 | - | 199+60 | In Place | Cutoff Wall to Elev. -5 ft (45 ft Deep) |
| 199+60 | - | 215+30 | In Place | Cutoff Wall to Elev. -80 ft (110 ft Deep) |
| 215+30 | - | 307+60 | In Place | None |

**Table 13-3 – Geotechnical Recommendations – Port North Levee**

| Port North Levee | | | | |
|---|---|---|---|---|
| Station | | | Levee | Recommended Improvements |
| From | - | To | | |
| 0+00 | - | 245+65 | In Place | None |

**Table 13-4 – Geotechnical Recommendations – Yolo Bypass North – East Levee**

| Yolo Bypass North - East Levee | | | | |
|---|---|---|---|---|
| Station | | | Levee | Recommended Improvements |
| From | - | To | | |
| 0+00 | - | 25+00 | In Place | None |
| 25+00 | - | 50+00 | In Place | Cutoff Wall to Elev. -10 ft (40 ft Deep) |
| 50+00 | - | 65+00 | In Place | None |
| 65+00 | - | 111+35 | In Place | None |
| 111+35 | - | 136+00 | In Place | None |
| 136+00 | - | 155+00 | In Place | Cutoff Wall to Elev. -70 ft (100 ft Deep) |
| 155+00 | - | 197+55 | In Place | None |

## 13.2 SOUTH BASIN

Within the south basin of the project, the predominant recommended fix is a cutoff wall and implementation is similar to the north basin. A notable variation is that on the Sacramento River levees, the recommendations could be constructed as fix-in-place using the existing footprint, adjacent levee to the existing embankments, or a setback levee. From discussion with the state and local sponsors, consideration is giving to including a setback or adjacent levee. This process will be detailed programmatically from a project perspective additional to the geotechnical concerns as USACE HQ approval is typically required. In conjunction with a cutoff wall, a seepage berm may be constructed as well to mitigate deep underseepage concerns. Other recommendations include relief wells and drained landside stability berms.

Common to the Sacramento River within the south basin, is a silty sand embankment underlain by an interbedded clay and silt blanket. The sand stringers interbedding the blanket pose uncertainty to potential development of seepage paths. Construction of a shallow keyed-in cutoff wall would mitigate against the development of the underseepage and through seepage gradients.

The tables below detail the extent and various combinations of the geotechnical recommendations per channel.

**Table 13-5 – Geotechnical Recommendations – Sacramento River South – West Levee**

| Sacramento River South - West Levee | | | | |
|---|---|---|---|---|
| **Station** | | | **Levee** | **Recommended Improvements** |
| **From** | **-** | **To** | | |
| 0+00 | - | 43+00 | Adjacent | Cutoff Wall to Elev. -5 ft (35 ft Deep) |
| 43+00 | - | 65+00 | Adjacent | Cutoff Wall to Elev. -5 ft (35 ft Deep) and Seepage Berm 70 ft wide |
| 65+00 | - | 167+00 | Setback Levee or Adjacent | Cutoff Wall to Elev. -5 ft (25 ft Deep) and Seepage Berm 80 ft wide |
| 167+00 | - | 275+00 | Setback Levee or Adjacent | Cutoff Wall to Elev. 0 ft (20 ft Deep) and Seepage Berm 100 ft wide |
| 275+00 | - | 295+00 | Adjacent | Cutoff Wall to Elev. -70 ft (100 ft Deep) |
| 295+00 | - | 315+00 | Setback Levee | None |
| 315+00 | - | 332+70 | In Place | None |
| South Extension | | | In Place or Adjacent | Cutoff Wall to Elev. -5 ft (40 ft Deep) with Landside Slope Flattening (from ±2:1 to 3:1) and Seepage Berm 80 ft wide |

**Table 13-6 – Geotechnical Recommendations – South Cross Levee**

| South Cross Levee | | | | |
|---|---|---|---|---|
| **Station** | | | **Levee** | **Recommended Improvements** |
| **From** | **-** | **To** | | |
| 0+00 | - | 5+00 | In Place | Landside Drained Stability Berm |
| 5+00 | - | 55+00 | In Place | Relief Wells with Screen Intervals From -9.5 to -23.5 and -39.5 to -58, Total Well Depth = 70 ft Spaced @ 50 ft |
| 55+00 | - | 65+00 | In Place | Landside Drained Stability Berm |

**Table 13-7 – Geotechnical Recommendations – Port South Levee**

| Port South Levee | | | | |
|---|---|---|---|---|
| Station | | | Levee | Recommended Improvements |
| From | - | To | | |
| 0+00 | - | 120+00 | In Place | None |
| 120+00 | - | 130+00 | In Place | Cutoff Wall to Elev. -55 ft (70 ft Deep) |
| 130+00 | - | 189+65 | In Place | None |

**Table 13-8 – Geotechnical Recommendations – Yolo Bypass South – East Levee**

| Yolo Bypass South - East Levee (Deep Water Ship Channel East Levee) | | | | |
|---|---|---|---|---|
| Station | | | Levee | Recommended Improvements |
| From | - | To | | |
| 0+00 | - | 15+00 | In Place | Cutoff Wall to Elev. -100 ft (120 ft Deep) |
| 15+00 | - | 85+55 | In Place | Cutoff Wall to Elev. -110 ft (130 ft Deep) |
| 85+55 | - | 145+00 | In Place | Cutoff Wall to Elev. -30 ft (50 ft Deep) |
| South Extension | | | In Place | Levee Degrade and Reconstruction with Landside Slope Flattening (from ±2:1 to 3:1) and Seepage Berm 80 ft wide and Relocate High Line Canal |

**Table 13-9 – Geotechnical Recommendations – Deep Water Ship Channel West Levee**

| Deep Water Ship Channel West Levee | | | | |
|---|---|---|---|---|
| Station | | | Levee | Recommended Improvements |
| From | - | To | | |
| 0+00 | - | 35+00 | In Place | Cutoff Wall to Elev. -60 ft (85 ft Deep) |
| 35+00 | - | 60+00 | In Place | None |
| 60+00 | - | 115+00 | In Place | Cutoff Wall to Elev. -60 ft (85 ft Deep) |
| 115+00 | - | 130+00 | In Place | None |
| 130+00 | - | 200+00 | In Place | Cutoff Wall to Elev. -30 ft |
| 200+00 | - | 290+00 | In Place | Cutoff Wall to Elev. -55 ft (75 ft Deep) |
| 290+00 | - | 1133+14 | In Place | None |

**Table 13-10 – Geotechnical Recommendations – Sacramento Bypass North Levee**

| Sacramento Bypass North Levee | | | | |
|---|---|---|---|---|
| Station | | | Levee | Recommended Improvements |
| From | - | To | | |
| 0+00 | - | 33+66 | New Levee | New Levee (20ft Crest Width 3:1 side slopes, inspection trench) with seepage berms 300ft wide. Or New Levee (20ft Crest Width 3:1 side slopes, with Seepage Berm 80ft in width and Cutoff Wall to El. -5ft (20ft deep)) |

## 14.0 PROBABILISTIC ANALYSIS

### 14.1 ANALYSIS METHODOLOGY

Index points were selected for geotechnical analysis to represent the critical surface and subsurface conditions of each planning reach in order to identify the geotechnical deficiencies of the reach. The sections were selected based on previous geotechnical analysis, past levee performance, existing levee improvements, subsurface data, laboratory test results, surface conditions, field reconnaissance, and levee geometry. The ground surface elevations used in the cross-sections were based on the LiDAR and bathymetric surveys. The analysis model stratigraphy was interpreted based on existing boring logs near the index point.

The First-Order-Second-Moment (FOSM) method, as recommended in ETL 1110-2-556, "Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies" dated 28 May 1999, was followed during the probabilistic evaluation of each index point. In this approach, the uncertainty in performance is taken to be a function of the uncertainty in model parameters. The standard deviations of a performance function were estimated based on the expected values (means) and the standard deviation of the random variable means. The performance functions considered were underseepage, through-seepage, and slope stability.

The final result of the FOSM method is a reliability index, Beta (β), representing the amount of standard deviation of the performance function by which the expected value exceeds the limit equilibrium state. The limit equilibrium state was defined using a factor of safety of 1.0. The standard deviation and variance of the performance function are calculated from the standard deviation and variance of the foundation and embankment parameters using the Taylor's series method based on a Taylor's series expansion of the performance function about the expected values. The partial derivatives were calculated numerically using an increment of plus and minus one standard deviation centered on the expected mean value. The variance of the performance function was obtained by summing the products of the partial derivatives of the performance function considering the variance of the corresponding parameters. The probability of poor performance Pr(f) of the levee was expressed as a function of the river water elevation and the random variables of each performance function.

Potential sources of levee distress or failure considered in the analyses were underseepage through the levee foundation, through-seepage through the levee embankment, and instability of the landside levee slope under steady state conditions. The levees were evaluated against the above mentioned performance modes at five different water surface elevations (loading conditions), which included; levee crest, levee crest minus three feet, half levee height, toe plus three feet, and landside levee toe where the probability of failure was considered to be zero. Using this method of selecting loading conditions the levee performance curves should represent probability of poor performance at multiple flood frequencies.

Sudden drawdown conditions may result in levee slope failure but it is unlikely to provide flooding of the area, the failure occurring when the water is at low elevation. Therefore this condition was not considered in the analysis. Additionally, a judgment based conditional probability of poor performance considering the existing and past erosion history of the levee and riverbank, maintenance, seepage/sand boils and sliding historical conditions, encroachments, vegetation on the levee slopes and within the levee critical area, animal burrows and other external damaging conditions were included in the risk and uncertainty analysis.

The probability of poor performance was evaluated by assessing the foundation and embankment materials and assigning values for the probability moments of the random variables considered in the analyses. Random variables for underseepage included the ratio of the horizontal permeability of the aquifer to the vertical permeability of the blanket, blanket thickness, and aquifer thickness. Random variables for through-seepage included critical tractive stress, porosity, and intrinsic permeability of the levee embankment material. Random variables for slope stability included effective friction angle, effective cohesion, and total unit weight of the levee embankment, and effective friction angle and cohesion of the foundation material.

It should be noted that poor performance can potentially range in description and severity. This range may include initiation of failure modes which can lead to minimal consequences, which could include seepage with no material being transported or surface slope sloughing. Conversely poor performance can also include levee failure due to slope stability, underseepage, and breach all of which pose a threat to the integrity of the levee during a flood event.

### 14.1.1 UNDERSEEPAGE

Underseepage analysis was performed using the blanket theory analysis (BTA) as described in the Corps ETL 1110-2-556, EM 1110-2-1913, and TM 3-424. Finite element analyses using the SEEP2D program, part of the GMS version 6.5 software package, were developed to independently check the blanket theory results. In general, the finite element and the empirical seepage calculations supported each other, predicting qualitatively similar results. Statistical analysis was used for each reach in determination of the coefficients of variation and standard deviation of the permeability ratios, blanket thickness and thickness of the underlying aquifer. A critical gradient of 0.80 was used, corresponding to 112pcf unit weight of the blanket. The unit weight of the blanket was considered the same at all index points. Values of vertical and horizontal permeabilities based on material classification and fines content are shown in Table 18-1 below and are based on the many past and ongoing geotechnical studies within the project area.

In comparison to the deterministic analysis which accounts for the most critical geotechnical conditions, the probabilistic analysis methodology accounted for potential subsurface material variations in the project reach in the vicinity of the cross section, and denoted a transformed blanket thickness and associated aquifer thickness using a number of borings near and at the project cross section. As a result, it may be possible that the transformed blanket thickness carried forward into the blanket theory calculation for underseepage gradients was greater than the deterministic value. This difference may yield opposing results in comparison between probabilistic and deterministic evaluations.

**Table 14-1: Vertical and Horizontal Hydraulic Conductivity**

| Material Type | Soil Description | Hydraulic Conductivity | | | | |
|---|---|---|---|---|---|---|
| | | $K_H$ (cm/sec) | $K_H$ (ft/day) | $K_H/K_V$ | $K_V$ (cm/sec) | $K_V$ (ft/day) |
| Clay | Blanket ≥10ft Thick | 1.0E-05 | 0.028 | 4 | 2.5E-06 | 0.0071 |
| | Blanket 5ft<>10ft Thick | 1.0E-05 | 0.028 | 1 | 1.0E-05 | 0.028 |
| | Blanket ≤5ft Thick | 1.0E-05 | 0.028 | 0.1 | 1.0E-04 | 0.28 |
| Silt | Elastic (plastic) | 5.0E-05 | 0.14 | 4 | 1.3E-05 | 0.035 |
| | Non-plastic | 2.0E-04 | 0.57 | 4 | 5.0E-05 | 0.14 |
| Clayey Sand to Sand | 30-49% fines | 5.0E-05 | 0.14 | 4 | 1.3E-05 | 0.035 |
| | 13-29% fines | 1.0E-04 | 0.28 | 4 | 2.5E-05 | 0.071 |
| | 8-12% fines | 1.0E-03 | 2.8 | 4 | 2.5E-04 | 0.71 |
| | 0-7% fines | 5.0E-03 | 14 | 4 | 5.0E-04 | 3.5 |
| Silty Sand to Sand | 30-49% fines | 5.0E-04 | 1.4 | 4 | 1.3E-04 | 0.35 |
| | 13-29% fines | 1.0E-03 | 2.8 | 4 | 2.5E-04 | 0.71 |
| | 8-12% fines | 5.0E-03 | 14 | 4 | 5.0E-04 | 3.5 |
| | 0-7% fines | 1.0E-02 | 28 | 4 | 1.0E-03 | 7.1 |

**Table 14-1: Vertical and Horizontal Hydraulic Conductivity (continued)**

| Material Type | Soil Description | Hydraulic Conductivity | | | | |
|---|---|---|---|---|---|---|
| | | $K_H$ (cm/sec) | $K_H$ (ft/day) | $K_H/K_V$ | $K_V$ (cm/sec) | $K_V$ (ft/day) |
| Gravel | 28-49% fines | 4.0E-04 | 1.13 | 4 | 1.0E-04 | 0.28 |
| | 18-27% fines | 1.0E-03 | 2.8 | 4 | 2.5E-04 | 0.71 |
| | 13-17% fines | 6.0E-03 | 17 | 4 | 6.0E-04 | 4.3 |
| | 8-12% fines | 1.2E-02 | 34 | 4 | 1.2E-03 | 8.5 |
| | 0-7% fines | 2.5E-02 | 71 | 4 | 2.5E-3 | 17.8 |

### 14.1.2 THROUGH SEEPAGE

Levees constructed either of fine grained clays, having stability berms with drainage layers extended along the levee slope that captures any seepage through the levee, or having cutoff walls constructed through the levee embankment are unlikely to be susceptible to through-seepage caused internal erosion. Levees of silt, silty sand, and sand were considered to be susceptible to internal erosion and were evaluated using the modified Khilar, Folger, and Gray internal erosion model as prescribed in ETL 1110-2-556. Using this method the critical gradient through the levee embankment was calculated based on variations in the critical tractive stress, porosity, and intrinsic permeability of the levee material and compared with the predicted horizontal gradient through the levee embankment from the SEEP2D model. Table 14-2 shows the mean values of the random variables of the levee embankment material used to calculate the critical gradient were critical tractive stress (dynes/cm$^2$) which was taken as ten times the $d_{50}$ (mm), the porosity based on material classification as proposed by Weight and Sonderegger in "Manual of Applied Hydrology", and intrinsic permeability was taken as approximately 1x10$^{-5}$ times the horizontal permeability (cm/sec). Table 14-3 presents coefficients of variation for the through-seepage analysis random variables that were obtained using methodologies outlined in ETL 1110-2-556.

**Table 14-2: Through-Seepage Random Variables**

| Material | Tractive Stress (dynes/cm$^2$) | Porosity (%) | Intrinsic Permeability (cm$^2$) |
|---|---|---|---|
| Clay | 0.3 - 0.4 | 40 - 70 | 1.0E-10 |
| Silt | 0.5 - 0.7 | 35 - 50 | 2.0E-9 – 5.0E-10 |
| Sand | 1.0 - 3.0 | 25 - 50 | 1.0E-6 – 5.0E-9 |
| Gravel | Not Used | 20 - 40 | 2.5.0E-6 – 4.0E-9 |
| Sand and Gravel | Not Used | 15 - 35 | |

**Table 14-3: Variation of Through-Seepage Random Variables**

| Random Variable | Coefficient of Variation (%) |
|---|---|
| Critical Tractive Stress ($T_c$) | 10 |
| Porosity (n) | 10 |
| Intrinsic Permeability ($K_o$ cm$^2$) | 30 |

### 14.1.3 LANDSIDE SLOPE STABILITY

The cases analyzed for stability risk analyses considered long-term conditions with steady state seepage along the landside slope of the levee. The phreatic surface and pore water pressures for the different water surface elevations were developed for the steady state condition using the SEEP2D finite element computer program developed as part of the GMS, version 6.5. The limit equilibrium computer program UTEXAS4 was used to perform the stability analyses. Circular failure surfaces were assumed and the embankment was modeled as homogeneous. All analyses consisted of running a search routine to identify the critical failure surface using the Spencer's Method.

A sensitivity study was done to determine which parameters in the slope stability calculations were most influential. For this study, the considered variables are soil strength and unit weights of the soil in the levee embankment and soil strength in the foundation. Statistical descriptors for these variables were determined using available site-specific information and published statistical data. The piezometric lines or pore water pressures for each water elevation were determined using the finite element program SEEP2D for the levee embankment and its foundation.

Soil strength parameters used in the stability analyses were the drained soil parameters, as shown in Table 14-4. The values in Table 14-4 were based on a generalized conservative assumption of shear strength by soil type from previous studies in the project area. For each index point the generalized assumption was compared with available field and laboratory testing from nearby explorations. The coefficients of variation for soil strength parameters and unit weight of the fill material in the levee or the top impervious blanket are shown in Table 14-5 and were obtained using methodologies outlined in ETL 1110-2-556, and those proposed by Harr in the

"Reliability-Based Design in Civil Engineering", and Duncan in the "Manual for Geotechnical Engineering Reliability Calculations".

**Table 14-4: Drained Shear Strength of Soil**

| Material Type | Soil Description | Shear Strength | | |
|---|---|---|---|---|
| | | $C'$ | $\Phi'$ ($^o$) | $\gamma$(pcf) |
| Cutoff Wall | SCB, SB, CB | 50 | 0 | 85 |
| Clay | CH Levee Embankment | 100 | 22 | 115 |
| | CH Foundation | 100 | 26 | 115 |
| | CL Levee Embankment | 50 | 24 | 115 |
| | CL Foundation | 50 | 28 | 115 |
| Silt | ML Levee Embankment- | 0 | 28 | 115 |
| | ML Foundation | 0 | 30 | 120 |
| Clayey Sand and Silty Sand | - | 0 | 33 | 125 |
| Sand | - | 0 | 35 | 130 |
| Gravel and Drain Rock | - | 0 | 35 | 135 |

**Table 14-5: Variation of Drained Shear Strength Parameters**

| Random Variable | Coefficient of Variation (%) |
|---|---|
| Effective Friction Angle (Φ) | 13 |
| Effective Cohesion (c psf) | 40 |
| Total Unit Weight (γ pcf) | 7 |

### 14.1.4 JUDGMENT

A judgment based conditional probability function was based on existing conditions of the levee such as encroachments on the levee slopes, vegetation on the levee slopes, existing cracks and holes due to animal burrows, and based on the past history of sand boils, or slope failures. Generally, past experience with poor performance at utility crossing and rodent activity indicates the risk of failure is somewhat significant in the analyzed areas. The judgment based curve is included for each analyzed levee cross section and in the combined curve of failure.

In June 2009, an expert elicitation was conducted for the purpose of developing the geotechnical judgment portion of the curves for the American River Common Features project, the meeting minutes are included as Enclosure 6. In relation to physical location, both the American River Common Features and West Sacrament Project are in close proximity to one another, lying on both the east and west of the Sacramento River. The findings of the expert elicitation were considered to be applicable as similar conditions are present in the West Sacramento Project area. The expert elicitation was conducted in accordance with ETL 1110-2-561, "Appendix E, Expert Elicitation in Geological and Geotechnical Applications" 31 January 2006. The members of the expert elicitation team were highly recognized professional specialists, representing the Reclamation Districts managing and operating the levee system, and specialists in erosion and in geotechnical issues. The expert elicitation focused on the judgment part of the geotechnical risk and uncertainty curves for the flood control structures. The expert elicitation was conducted over a three-day period in which the most representative reaches of each basin of the study were discussed. The expert elicitation team discussed and reached consensus on the impact of different factors of the judgment curve, such as:

a) The vegetation on the levees and within the levee right of way
b) Penetrations through the levee and foundation
c) Encroachments into the levee and levee right-of-way
d) Erosion of the riverbank and waterside slopes of the levee
e) Animal burrows

The conclusion reached by the panel was that the probability of poor performance, as a function of stage of the river, may be reduced by 50% when the river reached 4-5 feet above the landside toe, by 30% when the river stage is up to 8-9 feet above the landside levee toe, and by 10% when the river reaches 11-12 feet above the landside toe. This conclusion was considered to be applicable to each of the contributing factors on the judgment curve and the probabilities adjusted accordingly.

### 14.1.5 COMBINED CURVES

The total conditional probability of poor performance as a function of floodwater elevation has been developed by combining the probability of failure functions for four failure modes; underseepage, through-seepage, slope instability, and judgment.

## 14.2 LEVEE PERFORMANCE CURVES

The results of the geotechnical risk and uncertainty analyses are briefly discussed in the following sections. As previously discussed, underseepage, through seepage, and slope stability probabilities of poor performance were calculated analytically based on site specific subsurface information used to select material parameters and coefficients of variation. Included as Enclosure 4 are the spreadsheet analyses used to calculate the probabilities of poor performance, these spreadsheets include data from borings used to select parameters, the selected parameters, and the calculated results. The judgment curve remains as the non analytical component to the curve, those probabilities of failure were based on site specific conditions regarding vegetation, penetrations, encroachments, erosion and animal burrows. The reach description section of this report described in general terms the levee conditions regarding vegetation, penetrations, encroachments, and animal burrows. The erosion section of this report described the general erosion conditions for each reach. It should be noted that the subsurface conditions are compiled using geotechnical investigations at and adjacent to the analysis section and it may conclude that a variation in description of the subsurface is present when compared to the deterministic analysis section which accounts for the most critical geotechnical conditions. As such, the results may differ with respect to one another probabilistically and deterministically.

### 14.2.1 NORTH BASIN – SACRAMENTO RIVER WEST LEVEE – STA. 96+00

Borings chosen to be used in probabilistic analyses resulted in a mean blanket thickness value of 23.0 ft with a coefficient of variation of 17, and a mean aquifer thickness of 58.0 ft with a coefficient of variation of 12. The blanket was comprised of predominantly silts and lean clays. The aquifer was made up of poorly graded sands.

Probabilistic analyses resulted in potential poor performance due to landside slope stability yielding a Pr(f) of 93.7% at the crest. The without project judgment based probability portion of the curve was comprised mainly of erosion, and encroachments, accounting for 20.0% and 3.0% respectively at the crest. Overall judgment based contributions account for a Pr(f) of 24.7% of the without project combined curve at the levee crest. Figure 14-1 presents the without project conditions combined curve.

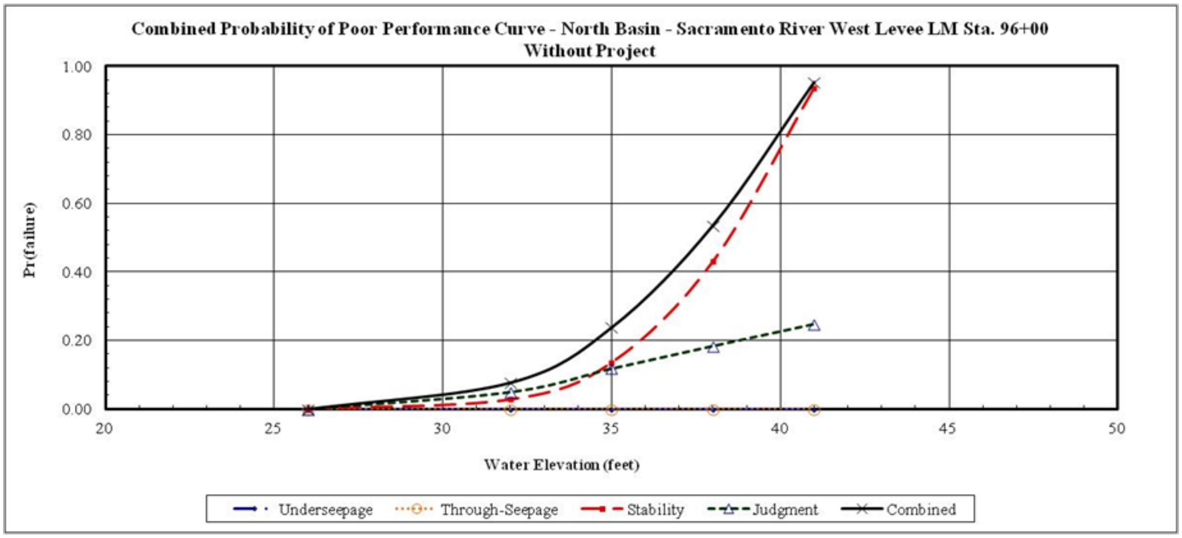



**Figure 14-1: Combined Probability of Poor Performance for Without Project Conditions**

With project improvement measures reduce judgment based probability due to erosion to a Pr(f) of 2.0% by placing rip rap erosion protection, and mitigate slope stability at the levee crest. Additionally, incorporation of a cutoff wall in this location addresses excess pore water pressure that lead to landside levee slope instability. Figure 14-2 presents the with project conditions combined curve.

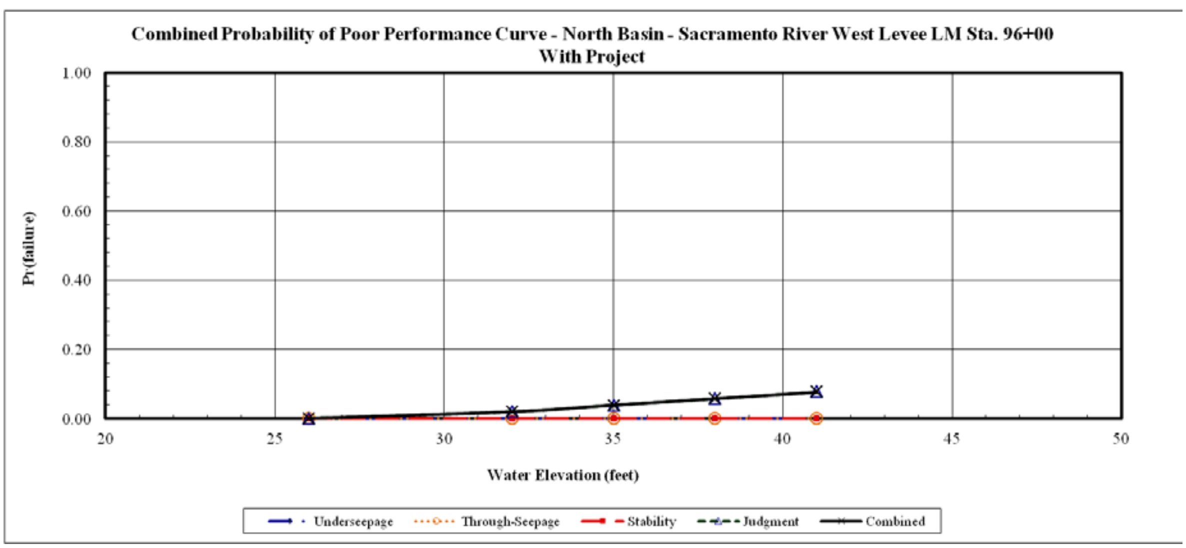



**Figure 14-2: Combined Probability of Poor Performance for With Project Conditions**

### 14.2.2 NORTH BASIN – SACRAMENTO RIVER WEST LEVEE – STA. 190+00

Borings chosen to be used in probabilistic analyses resulted in a mean blanket thickness value of 10.0 ft with a coefficient of variation of 0, and a mean aquifer thickness of 63.0 ft with a coefficient of variation of 5. The blanket was comprised of predominantly silts and lean clays. The aquifer was made up of poorly graded silty sands.

Probabilistic analyses resulted in potential poor performance due to landside slope stability yielding a Pr(f) of 87.9% at the crest. The without project judgment based probability portion of the curve was comprised mainly of erosion, and encroachments, accounting for 20.0% and 3.0% respectively at the crest. Overall judgment based contributions account for a Pr(f) of 35.6% of the without project combined curve at the levee crest. Figure 14-3 presents the without project conditions combined curve.

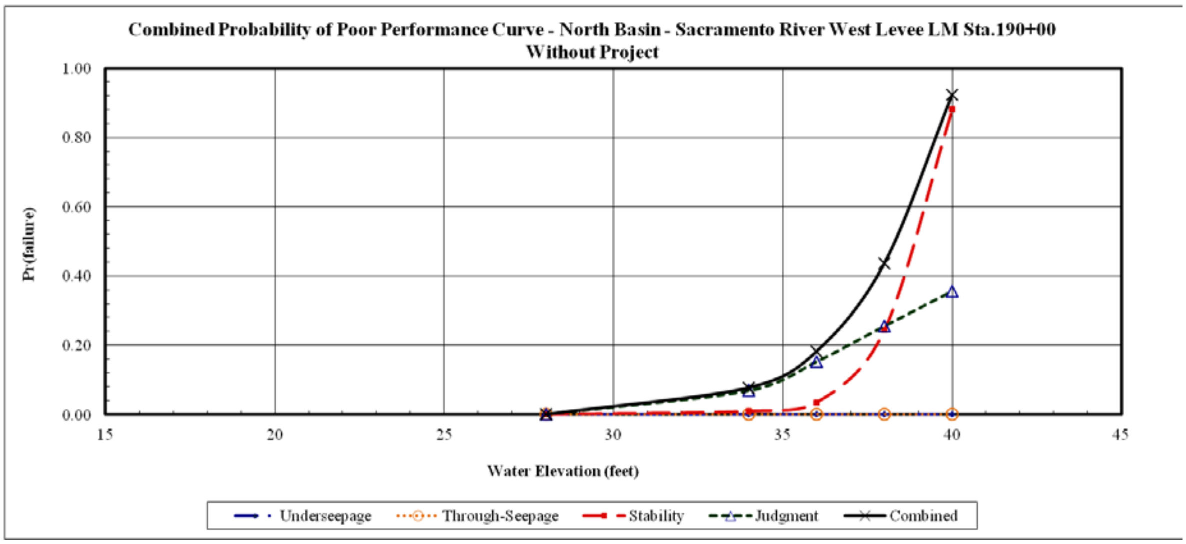



**Figure 14-3: Combined Probability of Poor Performance for Without Project Conditions**

With project improvement measures reduce judgment based probability due to erosion to a Pr(f) of 2.0% by placing rip rap erosion protection and mitigate slope stability at the levee crest and encroachments are reduced to a Pr(f) of 2.0%. The overall judgment based contribution account for a Pr(f) of 8.0%. Figure 14-4 presents the with project conditions combined curve.

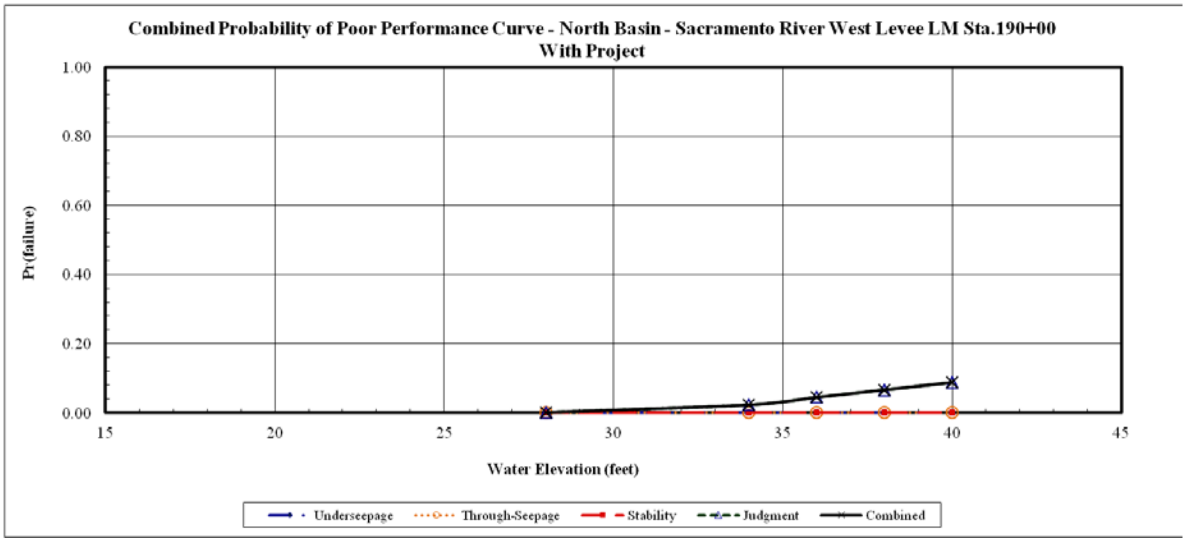



**Figure 14-4: Combined Probability of Poor Performance for With Project Conditions**

### 14.2.3 NORTH BASIN – YOLO BYPASS EAST LEVEE – STA. 107+31

Borings chosen to be used in probabilistic analyses resulted in a mean blanket thickness value of 22.0 ft with a coefficient of variation of 14, and a mean aquifer thickness of 27.0 ft with a coefficient of variation of 15. The blanket was comprised of predominantly fat clay. The aquifer was made up of poorly graded sand.

Probabilistic analyses resulted in potential poor performance due to underseepage and landside slope stability and yielding a Pr(f) of 99.57% and 88.7% at the crest respectively. The without project judgment based probability portion of the curve was comprised mainly of vegetation, and erosion, accounting for 5.0% and 4.0% respectively at the crest. Overall judgment based contributions account for a Pr(f) of 14.2% of the without project combined curve at the levee crest. Figure 14-5 presents the without project conditions combined curve.

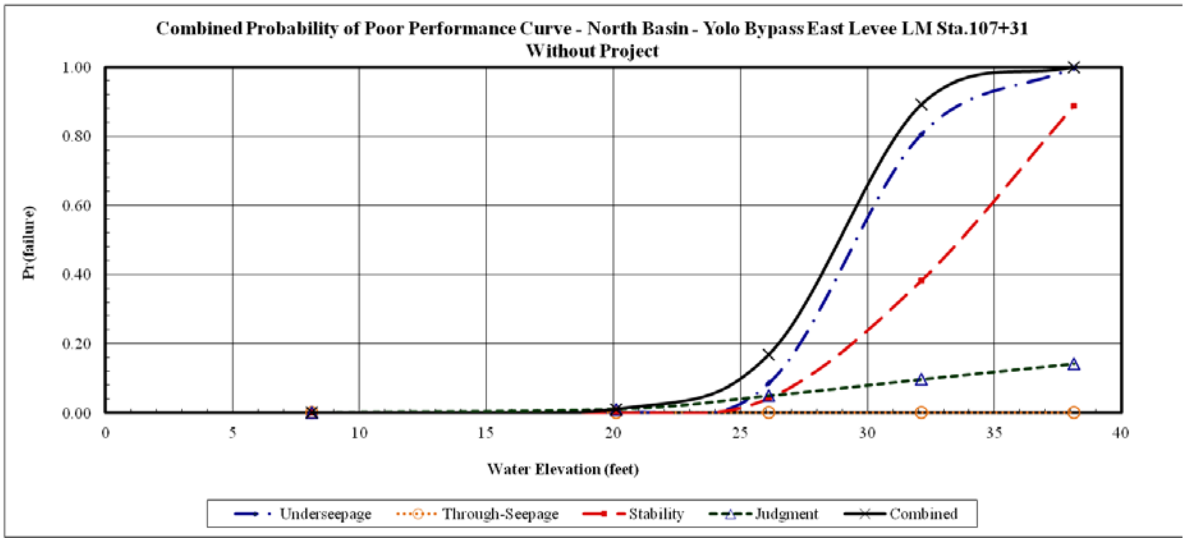



**Figure 14-5: Combined Probability of Poor Performance for Without Project Conditions**

With project conditions analyses were completed with the incorporation of embankment fill and drain. This improvement mitigated underseepage and landside slope stability concerns. With project improvement measures reduce judgment based probability due to vegetation to a Pr(f) of 1.0%. Figure 14-6 presents the with project conditions combined curve.

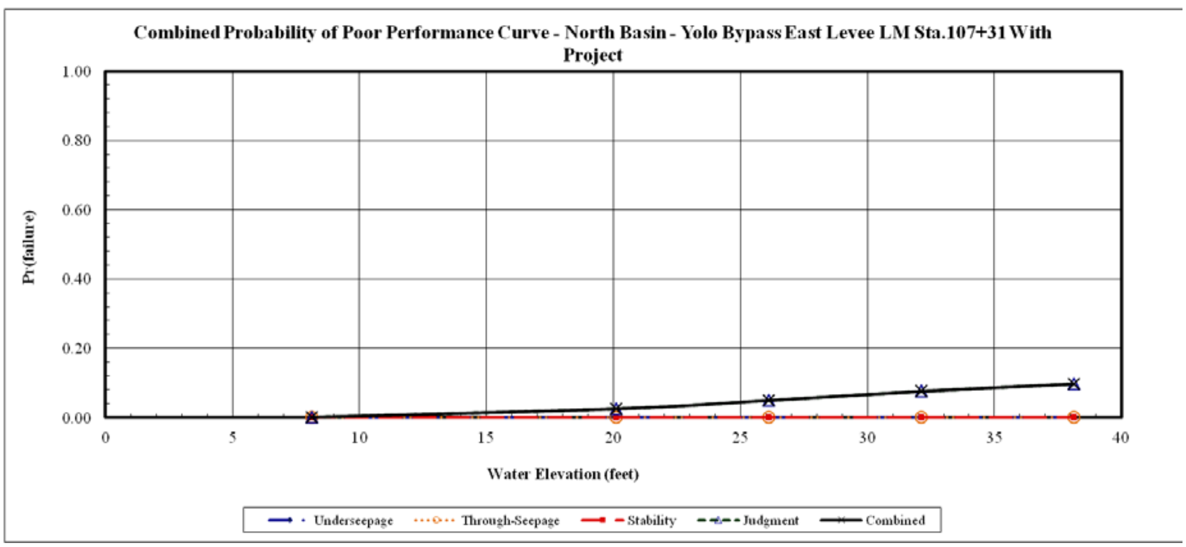



**Figure 14-6: Combined Probability of Poor Performance for With Project Conditions**

### 14.2.4 NORTH BASIN – SACRAMENTO BYPASS SOUTH LEVEE – STA. 52+00

Borings chosen to be used in probabilistic analyses resulted in a mean blanket thickness value of 36.0 ft with a coefficient of variation of 25, and a mean aquifer thickness of 36.0 ft with a coefficient of variation of 50. The blanket was comprised of predominantly lean clay. The aquifer was made up of poorly graded sands and well graded gravels.

Probabilistic analyses resulted in potential poor performance due to landside slope stability yielding a Pr(f) of 42.9% at the crest. The without project judgment based probability portion of the curve was comprised mainly of utilities, accounting for 5.0% at the crest. Overall judgment based contributions account for a Pr(f) of 5.0% of the without project combined curve at the levee crest. Figure 14-7 presents the without project conditions combined curve.

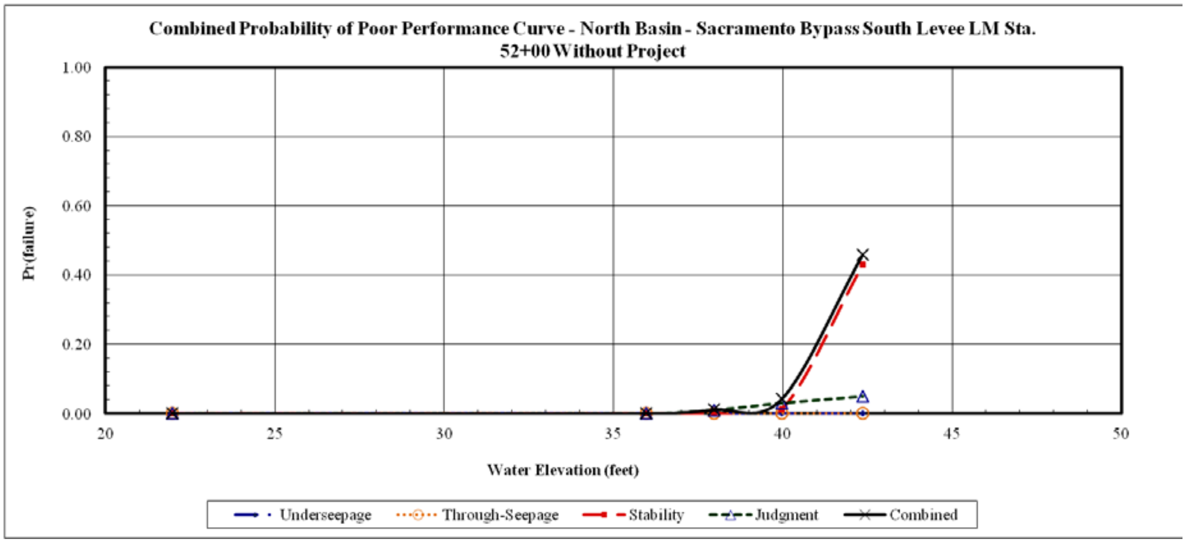



**Figure 14-7: Combined Probability of Poor Performance for Without Project Conditions**

With project conditions analyses were completed with the incorporation of a cutoff wall in this location to address excess pore water pressure that may lead to slope instability concerns of the landside levee slope. This improvement mitigated landside slope stability concerns. Figure 14-8 presents the with project conditions combined curve.

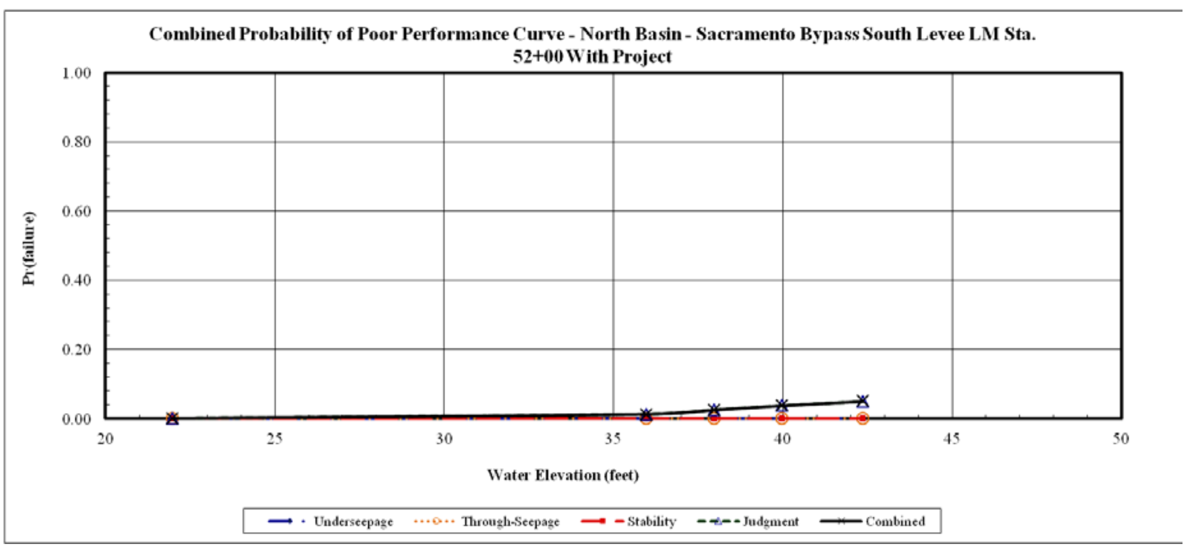



**Figure 14-8: Combined Probability of Poor Performance for With Project Conditions**

### 14.2.5 SOUTH BASIN – SACRAMENTO RIVER WEST LEVEE – STA. 264+00

Borings chosen to be used in probabilistic analyses resulted in a mean blanket thickness value of 16.0 ft with a coefficient of variation of 31, and a mean aquifer thickness of 50.0 ft with a coefficient of variation of 46. The blanket was comprised of predominantly lean clays and silts. The aquifer was made up of poorly graded sand and poorly graded silty sands.

Probabilistic analyses resulted in potential poor performance due to underseepage and landside slope stability and yielding a Pr(f) of 40.63% and 19.6% at the crest respectively. The without project judgment based probability portion of the curve was comprised mainly of vegetation accounting for 3.0% at the crest. Overall judgment based contributions account for a Pr(f) of 5.9% of the without project combined curve at the levee crest. Figure 14-9 presents the without project conditions combined curve.

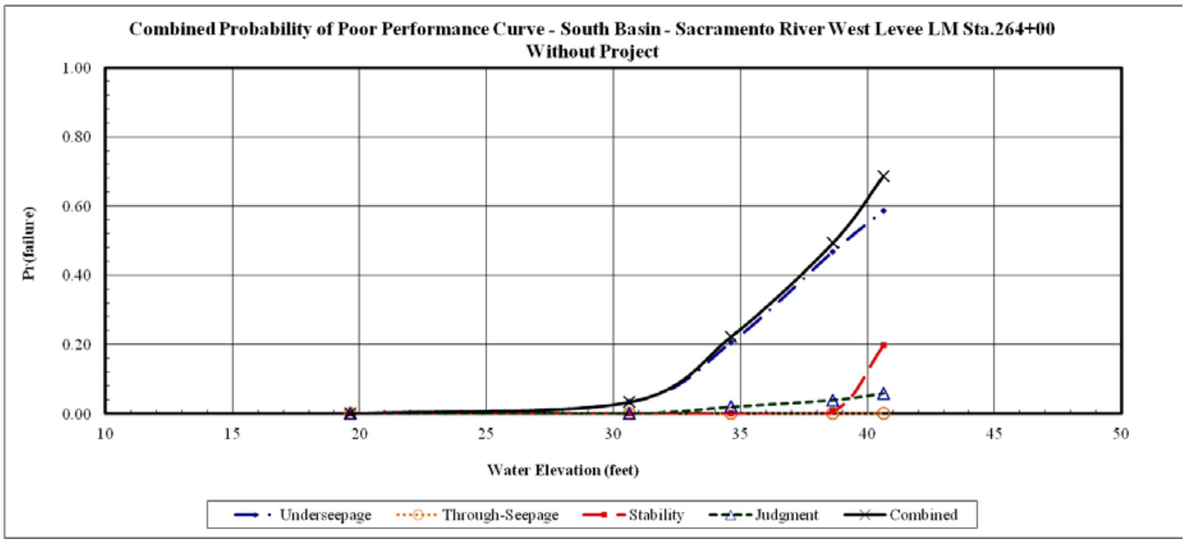



**Figure 14-9: Combined Probability of Poor Performance for Without Project Conditions**

With project conditions analyses were completed with the incorporation of an underseepage cutoff wall and seepage berm. These improvements mitigated underseepage and landside slope stability concerns. With project improvement measures reduce judgment based probability due to vegetation to a Pr(f) of 1.0%. Figure 14-10 presents the with project conditions combined curve.

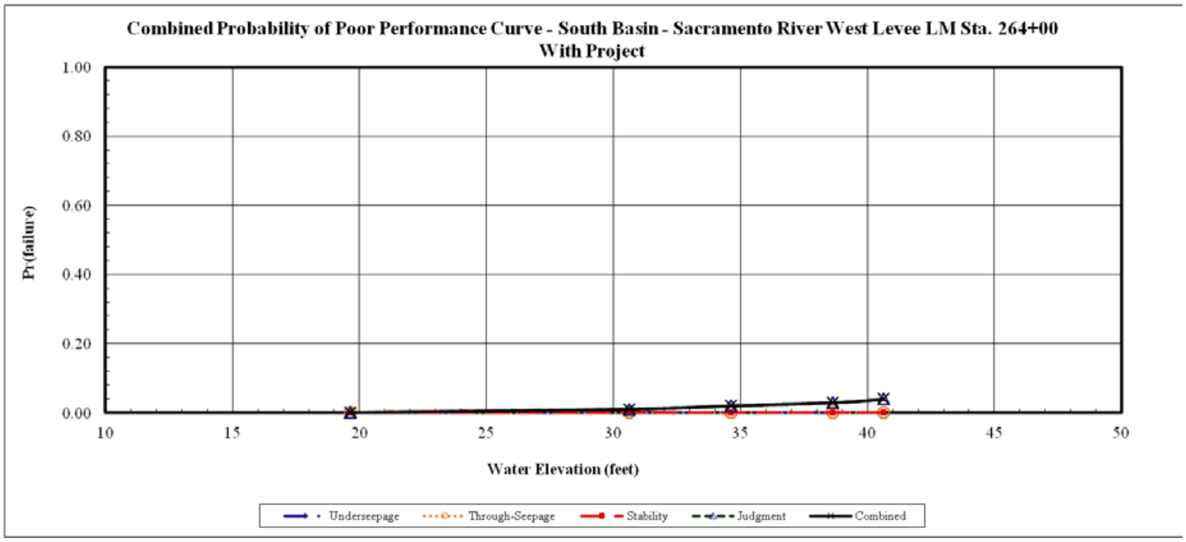



**Figure 14-10: Combined Probability of Poor Performance for With Project Conditions**

### 14.2.6 SOUTH BASIN – SACRAMENTO RIVER WEST LEVEE – STA. 80+00

Borings chosen to be used in probabilistic analyses resulted in a mean blanket thickness value of 24.0 ft with a coefficient of variation of 50, and a mean aquifer thickness of 39.0 ft with a coefficient of variation of 36. The blanket was comprised of predominantly silt. The aquifer was made up of poorly graded sand, poorly graded silty sands, and silty sand.

Probabilistic analyses resulted in potential poor performance due to underseepage yielding a Pr(f) of 9.6%. The without project judgment based probability portion of the curve was comprised mainly of vegetation accounting for 5.0% at the crest. Overall judgment based contributions account for a Pr(f) of 13.3% of the without project combined curve at the levee crest. Figure 14-11 presents the without project conditions combined curve.

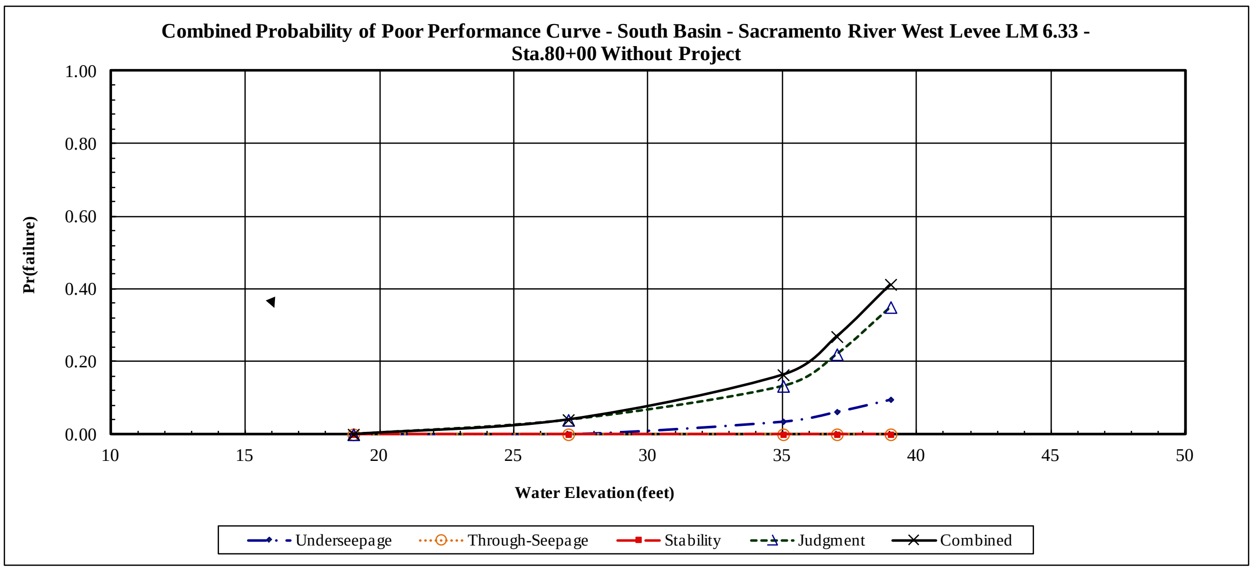



**Figure 14-11: Combined Probability of Poor Performance for Without Project Conditions**

With project conditions analyses were completed with the incorporation of an underseepage cutoff wall. These improvements mitigated underseepage and landside slope stability concerns by addressing excess pore water pressure that may develop leading to slope instability concerns of the landside levee slope. Figure 14-12 presents the with project conditions combined curve.

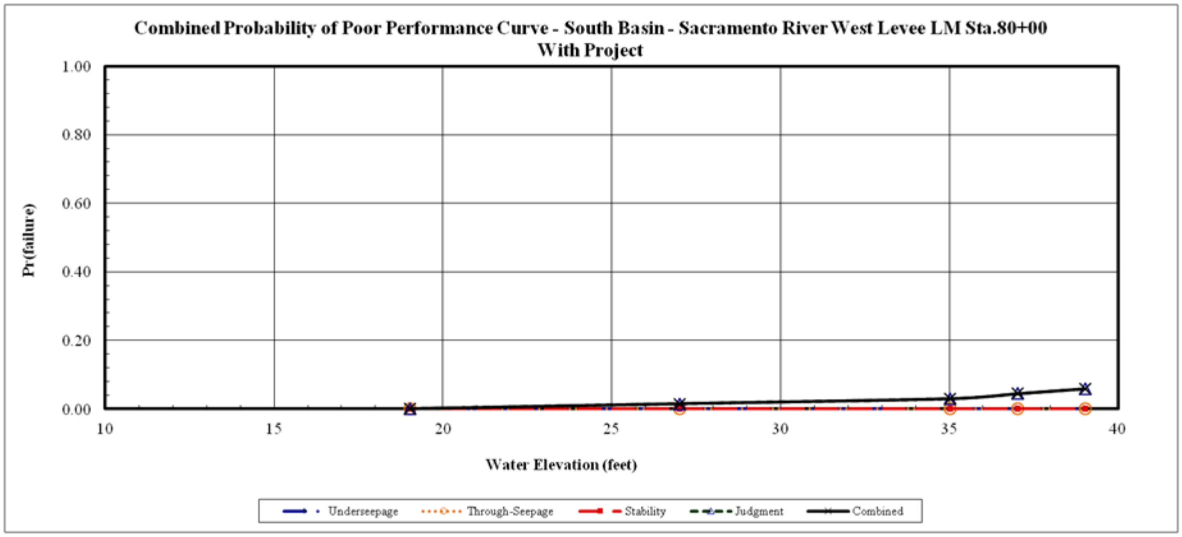



**Figure 14-12: Combined Probability of Poor Performance for With Project Conditions**

### 14.2.7 SOUTH BASIN – DEEP WATER SHIP CHANNEL WEST LEVEE – STA. 12+00

Borings chosen to be used in probabilistic analyses resulted in a mean blanket thickness value of 11.0 ft with a coefficient of variation of 18, and a mean aquifer thickness of 40.0 ft with a coefficient of variation of 10. The blanket was comprised of predominantly lean and fat clays. The aquifer was made up of poorly graded silty sands.

Probabilistic analyses resulted in potential poor performance due to underseepage and landside slope stability and yielding a Pr(f) of 99.0% and 3.0% at the crest respectively. The without project judgment based probability portion of the curve was comprised mainly of erosion accounting for 20.0% at the crest.  Overall judgment based contributions account for a Pr(f) of 35.0% of the without project combined curve at the levee crest. Figure 14-13 presents the without project conditions combined curve.

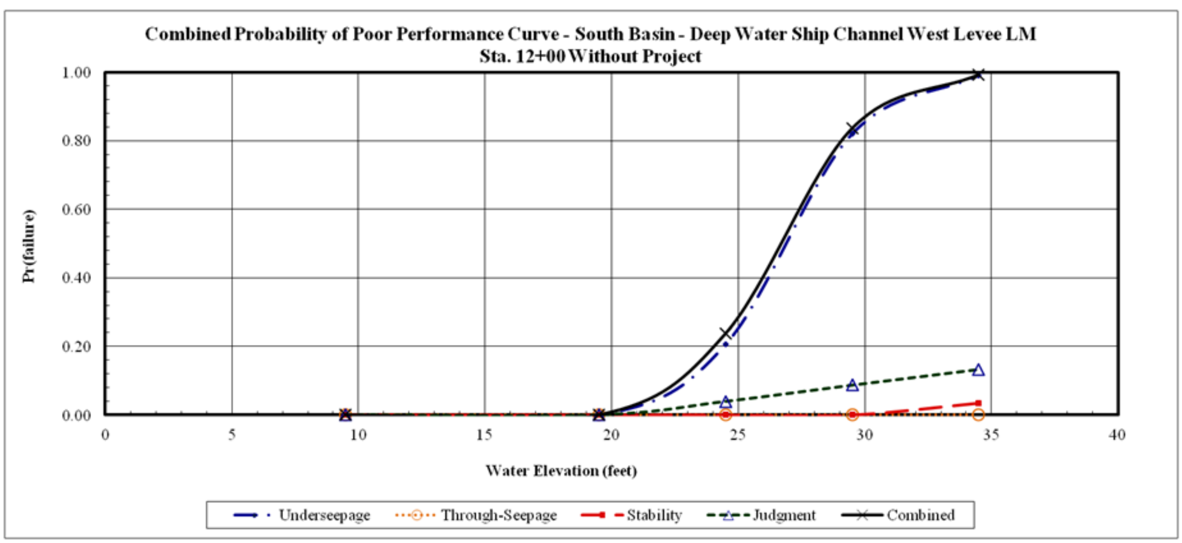



**Figure 14-13: Combined Probability of Poor Performance for Without Project Conditions**

With project conditions analyses were completed with the incorporation of an underseepage cutoff wall and seepage berm. These improvements mitigated underseepage and landside slope stability concerns. The remaining probability of failure was primarily attributed to the judgment based failure mode of erosion, is proposed to be mitigated through the placement riprap erosion protection. With project improvement measures reduce erosion to a Pr(f) of 2.0% at the levee crest. Figure 14-14 presents the with project conditions combined curve.

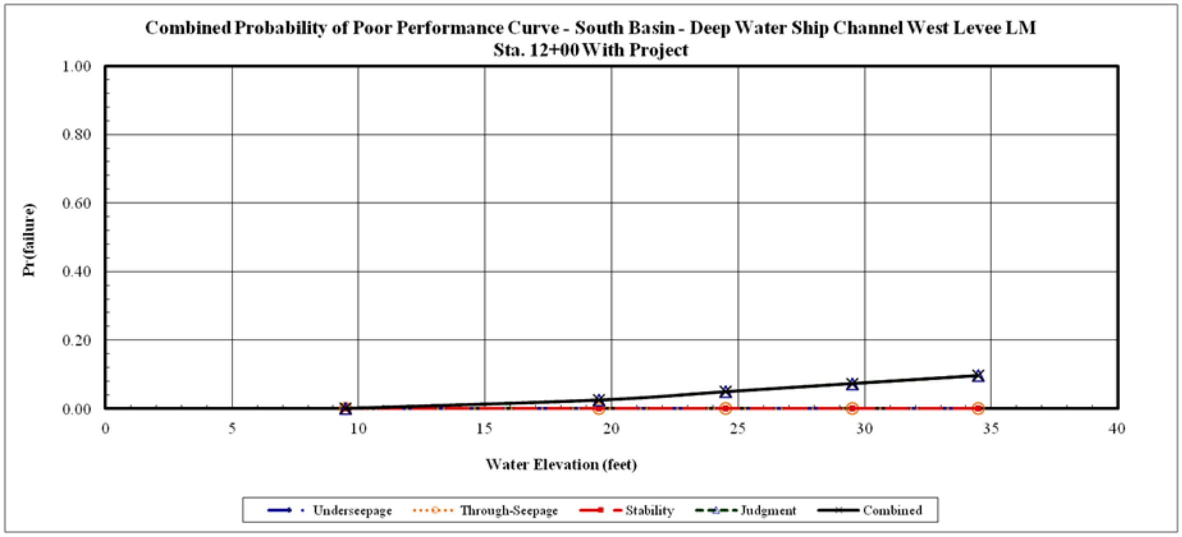



**Figure 14-14: Combined Probability of Poor Performance for With Project Conditions**

### 14.2.8 SOUTH BASIN – PORT SOUTH LEVEE – STA. 123+55

Borings chosen to be used in probabilistic analyses resulted in a mean blanket thickness value of 18.0 ft with a coefficient of variation of 67, and a mean aquifer thickness of 22.0 ft with a coefficient of variation of 14. The blanket was comprised of predominantly fat clays. The aquifer was made up of poorly graded sands.

Probabilistic analyses resulted in potential poor performance due to underseepage yielding a Pr(f) of 13.2% at the crest. The without project judgment based probability portion of the curve was comprised mainly of erosion accounting for 5.0% at the crest. Overall judgment based contributions account for a Pr(f) of 10.6% of the without project combined curve at the levee crest. Figure 14-15 presents the without project conditions combined curve.

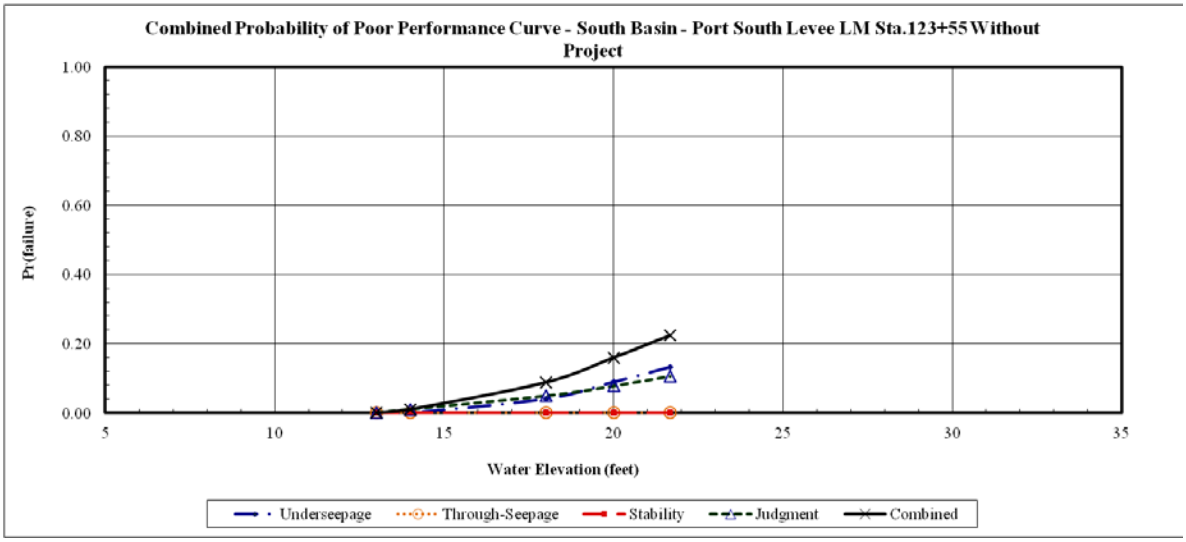



**Figure 14-15: Combined Probability of Poor Performance for Without Project Conditions**

With project conditions analyses were completed with the incorporation of an underseepage cutoff wall. These improvements mitigated underseepage concerns. The remaining probability of failure was primarily attributed to the judgment based failure mode of erosion, is proposed to be mitigated through the placement riprap erosion protection. Figure 14-16 presents the with project conditions combined curve.

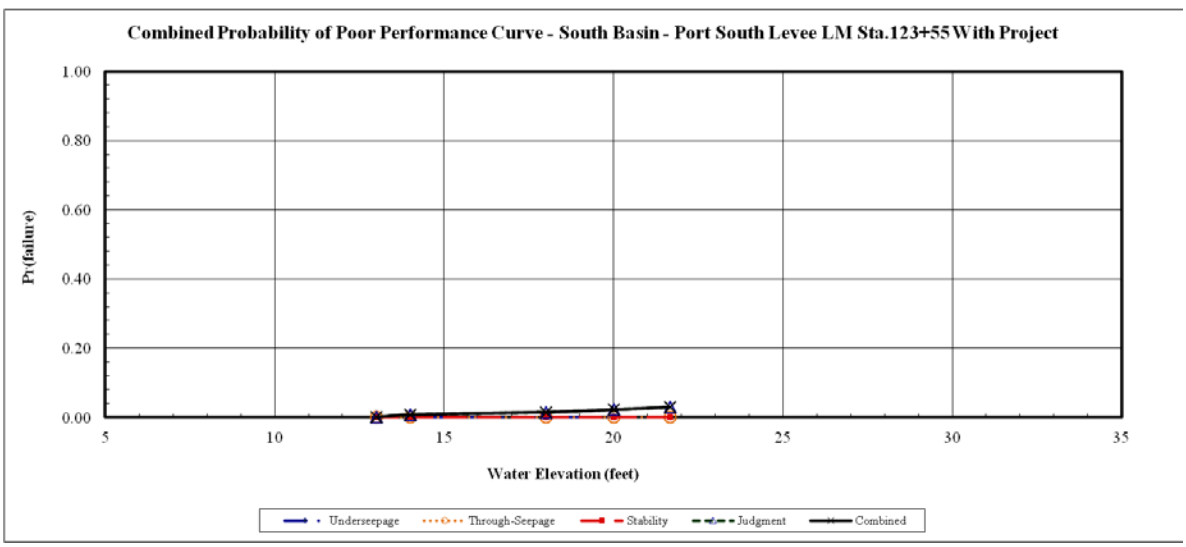



**Figure 14-16: Combined Probability of Poor Performance for With Project Conditions**

## 15.0 MATERIAL REQUIREMENTS

### 15.1 MATERIAL SPECIFICATIONS

It is anticipated that significant quantities of material will be required for construction of the proposed project. Several different improvement measures such as seepage berms, cutoff walls, embankment construction/reconstruction, and erosion protection are proposed. The following section describes proposed minimum material requirements.

#### 15.1.1 TYPE I LEVEE FILL (SELECT LEVEE FILL)

The Sacramento District, Geotechnical Engineering Branch, SOP-03 established the requirements of engineered fill to be used for the construction of the levee embankments. This is referred to as either Type I Levee Fill or as Select Levee Fill and meets the following requirements:

- 100% passing the 2-inch sieve
- minimum 20% fines content (silt and clay size particles)
- fines must have a liquid limit less than 45 and a plasticity index between 7 and 15
- no organic material or debris may be present

#### 15.1.2 RANDOM FILL

It is acknowledge that not all improvement features will require Type I Levee Fill and that a less stringent material specification is required for seepage berms, stability berms, and in some cases reconstructed embankment slopes. The actual specification of this material will be based on the type of material available at project borrow sites, but in general shall conform to the following requirements;

- 100% passing the 2-inch sieve
- minimum 12% fines content (silt and clay size particles)
- no organic material or debris may be present

#### 15.1.3 RIP-RAP

Since 1936 the Sacramento District has placed rock erosion protection on the banks and levees of the Sacramento River and associated tributaries. The SRBPP uses a standard rip-rap and filter gradation for repair sites which may be appropriate within the ARCF GRR study area. However, Civil Design Section A, Sacramento District calculated rip-rap requirements for a typical channel section with an average channel velocity of 7.0 fps and one for 12.0 fps. The resulting D100 were 18.0 and 36.0 inches with D15 of 7.1 and 14.3 inches respectively. The actual gradations will be determined during design but the rip-rap should be angular in shape, sound, durable, and hard. Rip-rap should also be free from laminations, weak cleavages, undesirable weather, or blasting or handling induced fractures. The rip-rap stone should be of such character that it will not disintegrate from the action of air, water, or the conditions of handling and placing and should be free from earth, clay, refuse, or adherent coatings.

## 15.2 ANTICIPATED BORROW SITES

As stated previously, significant quantities of engineered fill of various specifications and rock erosion protection will be required to construct the proposed project. The material is expected to be sourced from several sites including; newly identified borrow sites within approximately 25 miles of the study area, existing borrow sites identified for the Natomas Basin by SAFCA, the DWSC dredge disposal area, the existing levees, and existing commercial sources. Test pits and laboratory testing on materials collected from were provided by SAFCA as part of the NLIP borrow sites established for the Natomas Basin. Additionally, the Sacramento District has studied the DWSC spoil areas as a borrow source several time in the past, and a discussion of that borrow source is included below. Typically projects constructed by the Sacramento District utilize commercial borrow sites near the project area.

### 15.2.1 DESKTOP REGIONAL BORROW STUDY

A desktop regional borrow study was performed to identify potential borrow sites, within 25 miles of the study area, where enough soil could be sourced to satisfy the project needs. This study was performed by obtaining National Resources Conservation Service (NRCS) National Cooperative Soil Survey (NCSS) data, sorting the NCSS data based on material classification and engineering properties, using aerial photographs to identify areas of open or agricultural land, and then merging the sorted NCSS data with the open or agricultural land areas to obtain locations, acreage, and volume of potential borrow sites.

The NCSS is a nationwide partnership of federal, regional, state and local agencies; and private entities and institutions, led by the NRCS for the USDA, that work together to cooperatively investigate, inventory, document, classify, interpret, disseminate, and publish information about soils of the United States. The NCSS data was obtained from the Soil Data Mart, http://soildatamart.nrcs.usda.giv, in the Soil Survey Geographic Database (SSURGO) format for Placer, Sutter, Sacramento, Yolo, and Solano Counties. This data set consisted of georeferenced digital map data (polygons of soil map unit [MUSYM] boundaries) and computerized attribute data (engineering properties, agricultural properties, etc). The MUSYM were linked to attributes in a relational database, which gave the proportionate extent of the component soils and their properties. The NCSS data delineated the MUSYM (typically several named soils) into specific depth horizons (layers) giving soil properties to each horizon. The NCSS data was reduced to only those units and horizons which met material requirements for Type I Levee Fill.

After merging the polygons of NCSS MUSYM that met Type I Levee Fill requirements with polygons representing areas of open or agricultural land, acreages of potential borrow sites could be calculated from the coincident polygons. To obtain an approximate available volume for each of the potential borrow sites, a thickness of suitable material had to be chosen. The reduced NCSS data was sorted by thickness and MUSYM and split into two groups, units with greater than or equal to 30-inches and units with less than 30-inches of suitable thickness. The first group was termed to have a high confidence in obtaining Type I Levee Fill and the second group was termed as having low confidence in obtaining Type I Levee Fill. The mean thickness of the high confidence group was 42-inches and the mean thickness of the low confidence group was 12-inches. A shrinkage of 30% was assumed given potential transportation loss and assuming a relative compaction of 85% of the native materials at the borrow site. Volumes were then calculated in million cubic yards (MCY) for each group. The total available quantity of potential soil borrow was calculated to be 212 MCY over 105,000 acres. Plates 6 and 7 show the high confidence and low confidence areas of potential borrow sites.

In subsequent design phases, futher detailed analysis efforts encompassing greater vertical depths, of greater than 3 feet, will be considered with respect to borrow.

### 15.2.2 FISHERMAN LAKE COMPLEX BORROW SITE

The borrow site is located south of Del Paso Road, north of Radio Road and east of Power Line Road, about 400 feet east of the proposed landside levee toe in the vicinity of the Pumping Plant No. 3. The area is near the historical Fisherman Lake and is reclaimed for agricultural purpose. This borrow site will be used for construction of the adjacent levee landside of the existing levee and for the seepage berms on the landside levee slope on the east bank of the Sacramento River and north bank of the American River levee remediation. The materials found in the proposed borrow area contains clays with low and high plasticity, silts and some sandy clays and silts.

### 15.2.3 SOUTH SUTTER BORROW SITE

The borrow site is located east of the Sacramento River East Levee, north of Elkhorn Boulevard, south of Teal Bend, west of the Sacramento International Airport, at approximate 500 feet from the levee landside toe. The material in this borrow area consists of lean clays, lean clays with sand, some high plasticity clays, silts and sandy silt, and poorly graded sand. The material from this borrow area may be used for the adjacent levee and seepage berms along the Sacramento River east bank levee, with the condition that the high plasticity berm is used only in the working platform for the seepage cut-off wall. The area is mainly agricultural land within 2 miles from the Sacramento Airport which regulates the land use. Special approval and conditions are required by the Federal Aviation Administration to be respected if the borrow area is used.

### 15.2.4 NORTH AIRPORT BORROW SITE

The North Airport borrow site is located about a half of mile east of the Sacramento River east bank levee, north of the Sacramento International Airport. The area is also located within 2 miles from the Sacramento International Airport and consequently the same requirements of the Federal Aviation Administration should be meet if the borrow area is used. The borrow area is currently agricultural land and is designated as buffer lands for the Airport runway approaches, the purpose of it being to prevent land uses that are incompatible with Airport runways. Materials encountered in the borrow area consist of low plasticity clays, sandy clay, some higher plasticity clays, silty clay, sandy silt and clayey sand. The material may be used for the construction of the adjacent levee on the landside of the Sacramento River east bank levee and American River north bank levee and for the landside seepage berms.

### 15.2.5 BROOKFIELD BORROW SITE

The borrow site is located at the corner of the Pleasant Grove Creek Canal where it meets the Natomas Cross Canal within the Natomas basin, approximate 300 feet from the levee landside toe. The land is used for agriculture. Testing of the materials in the borrow area shows the material consisting of mainly low plasticity clay with less than 5 % of higher plasticity clay (with the LL less than 55), some sandy or silty clay and silts. The material may be used for remediation of the Natomas Cross Canal south bank levee and for the Pleasant Grove Creek Canal west bank levee.

### 15.2.6 TRIANGLE BORROW SITE

The borrow area is located east of the Natomas Basin, outside the protected area, south of the Natomas Cross Canal. This area is proposed to be used in case the material from the other borrow areas is insufficient. There were no sample collected from the area and no testing on the material. However, based on geomorphologic studies the material in the upper 5-10 feet is suitable for levee construction.

### 15.2.7 DEEP WATER SHIP CHANNEL BORROW SITE

The Deep Water Ship Channel (DWSC) navigation levee was constructed on the east side of the City of West Sacramento near the Yolo Bypass and has been used for disposal of dredged soils from the DWSC. This dredge disposal material placed on the waterside of the navigation levee has been proposed as a potential borrow source for several levee construction projects and was investigated for suitability of materials in July of 2009 by the Sacramento District and again in May of 2010 by Ayres and Associates for the Sacramento Districts. Both studies found that the majority of material is composed of highly plastic clays and silts and does not meet the requirements of SOP-003. Consequently, without some modification, such as lime or fly ash stabilization, the DWSC dredge disposal areas cannot be used for levee construction. Based on the 2010 Ayres and Associates report, it is projected that approximately 400,000 cubic yards of material is available at this borrow site.

### 15.2.8 COMMERCIAL BORROW SOURCES

Several privately owned and operated commercial soil borrow sites are located within approximately 30 to 50 miles of the study area, within the unincorporated area of Sacramento County. In general, they are located between Kiefer Boulevard to the north, Excelsior Road to the east, Elder Creek Road to the south and Hedge Avenue to the west. These borrow sites have supplied import fill material on various USACE projects in the past. While either the total or annually available material and its classification at the commercial sites cannot be defined with any certainty due to their private ownership, the sites typically utilized on USACE projects range in size from approximately 100 acres to 400 acres (all sites combined totaling approximately 950 acres, including aggregate sites) and contain sandy lean clay to clayey sand.

### 15.2.9 EXISTING LEVEE MATERIAL

Depending on the selected improvement measure, it is possible that existing levee material could be used as a source of borrow material. Typically, the existing levee is composed of poorly graded sands, silty sands, and sandy silts on the rivers and streams, while the bypass levees were constructed of fat clays. This material can be considered suitable for use in the construction of some stability berms, seepage berms, and for reconstructing the levee embankment where a cutoff wall with an impervious clay cap is proposed.

### 15.2.10 SOURCES OF RIP-RAP

A list of quarries is provided below that have been field-checked by the USACE and which have supplied specification rock on previous projects. Not all of the listed quarries have current test results available and complete testing of rock materials would be required during design.

COOL QUARRY
Located near Cool, CA
Holly Sugar
(560) 885-4244

SAN RAFAEL ROCK QUARRY
Located in San Rafael, CA
Dutra Material Corp.
(415) 459-7740

BANGOR QUARRY
Located near Bangor, CA
Roy E. Ladd Co.
(916) 241-6102

SPRING VALLY QUARRY
Located near Marysville, CA
Carl Woods, Co.
(530) 673-7877

TABLE MOUNTAIN QUARRY
Located near Jamestown, CA
George Reed, Inc.
(209) 984-5202

SNAKE CANYON QUARRY
Located in Napa, CA
Syar Industries, Inc.
(707) 252-8711

IONE QUARRY
Located near Ione, CA
Cal West Rock Products
(209) 274-2436

PARKS BAR QUARRY
Located near Marysville, CA
Nordic Industries
(530) 745-7124

JACKSON VALLEY QUARRY
Located near Ione, CA
George Reed, Inc.
(206) 984-5202

LAKE HERMAN QUARRY
Located near Vallejo, CA
Syar Industries, Inc.
(707) 252-8711

WOODS CREEK QUARRY
Located near Jamestown, CA
Sierra Rock Products
(209) 984-5307

HOGAN QUARRY
Located near Valley Springs, CA
Fort Construction Co.
(209) 333-1116

CARMICHAEL (VINA) QUARRY
Located near Vina, CA
Carl Woods Co.
(530) 673-7877

## 16.0 CONCLUSIONS

This report presented the results of geotechnical analyses and feasibility level design recommendations associated with the various alternatives under consideration to address technical deficiencies in the flood risk management system protecting the study area. The alternatives consisted of a combination of structural measures to mitigate deficiencies with levee height, geometry, erosion, access, vegetation, seepage, and slope stability.

The results of the without project seepage and slope stability analyses indicated that the levees in north basin including Sacramento River West Levee, Sacramento Bypass South Levee, and the Yolo Bypass East Levee along with the south basin including the Sacramento River West Levee, Port South Levee, South Cross Levee, Yolo Bypass East Levee, and the Deep Water Ship Channel West Levee did not meet seepage and/or stability requirements. The analyses showed that the levees did not meet criteria at varying flood frequencies typically between the 25 and 200 year events. The with project analyses typically included cutoff walls which resulted in the with project levee analyses satisfying criteria. It should be noted that the entire project area reaches on the aforementioned locations were not deficient; a percentage each of the project reaches exhibited a deficiency. Further detailed of the deficiencies and mitigation measures were displayed in Section 11.0 and Section 13.0. The recommended mitigation measures included in this report will be reconsidered when a further detailed design-level analysis is performed.

The results of the liquefaction triggering analysis and liquefaction-induced post-earthquake deformation based on limit equilibrium analysis indicated that liquefaction potential is likely at the Sacramento Bypass levees within the north basin and along both the Port South levee and Sacramento River West levee in the south basin. Moreover, at these locations, the analysis indicates that the post-earthquake deformation as the result of liquefaction of the material beneath the embankment is a global or structural failure mode that is very likely to compromise the ability to provide flood protection at these critical locations.

The without project levee performance curves indicate that the levees in North basin including the on Sacramento River West Levee, Yolo Bypass East Levee, and Sacramento Bypass South Levee, and within the South Basin including the Sacramento River West Levee, and Deep Water Ship Channel West Levee would perform unsatisfactorily when minimally to moderately loaded. In general, the analyses identified underseepage deficiencies and/or underseepage related slope stability deficiencies. Therefore, the with project levee performance curves typically included deep cutoff walls which resulted in significant reduction in probabilities of poor performance.

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